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STREAM FLOW IN GENERAL TERMS*

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WITH DISCUSSION BY MESSRS. FRANK S. BAILEY, CECIL E. PEARCE, F. THEODORE MAVIS, MORROUGH P. O'BRIEN, R. D. N. SIMHAM, AND MELVIN D. CASLER.

SYNOPSIS

The purpose of this paper is to present a workable general method for the analysis of stream flow in irregular channels—channels in which the invert slope and the channel cross-section are not constant. To attempt to apply the Chezy formula to such a case is to force a perfectly good formula into a situation to which it is not properly applicable. Various scattered attempts have been made to treat this question as applied to special cases, but no satisfactory general treatment has been found in such a form as to be readily followed by most computers.

An exhaustive consideration of the subject of "back-water curves" is contained in "Technical Reports," Part III, of The Miami Conservancy District, but this discussion is confined to "rectangular channels the width of which is uniform and so great in comparison with the depth that the hydraulic radius may be considered equal to the depth."

The discussion contained in this paper is general, and demonstrations of its applicability to weirs, orifices, and siphons are appended.

The Chezy formula, $v = C \sqrt{r\, s}$, is the one most commonly used in stream-flow computations. The factor of longitudinal slope, s, is sometimes referred to as the slope of the water surface and sometimes as the slope of the bed of the stream. As a matter of fact, it is neither of these except under special conditions. The Chezy slope factor should be defined as the friction factor, or rate of loss of head. This is equal to the slope of the water surface only when the velocity is constant at successive cross-sections and is equal to the slope of the stream bed only when "depth of water plus velocity head" is constant, as demonstrated subsequently.

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In the following discussion, the word, "channel", applies to any form of watercourse—canal, conduit, flume, pipe, or stream valley—flowing partly full, or full, or under pressure; and the formulas presented are applicable to all the cases mentioned.

NOMENCLATURE

The following nomenclature has been used:

Let w = unit weight of water, in pounds per cubic foot.

Q = quantity of flow, in cubic feet per second.

W = quantity of flow, in pounds per second $= w\,Q$.

a = cross-sectional area of stream, in square feet.

d = maximum depth of water from channel invert to free water surface at the section considered (measured to hydraulic gradient in case of full conduit), in feet.

b = width of channel at water surface (= zero in case of full conduit), in feet.

p = wetted perimeter, in feet.

r = hydraulic radius $\left(= \frac{a}{p}\right)$, in feet.

v = mean velocity $\left(= \frac{Q}{a}\right)$, in feet per second.

g = gravity factor (= 32.16).

C = Chezy coefficient.

$m = \frac{1}{2\,g\,a^2}$.

$n = \frac{p}{a^3\,C^2}$.

h = effective velocity head $\left(= \frac{v^2}{2\,g} = \frac{Q^2}{2\,g\,a^2} = m\,Q^2\right.$ if the velocity is uniform throughout the area, $\left.a\right)$.

i = invert drop in direction of flow per running foot of channel, in feet. (If invert rises in direction of flow, i is negative).

s = drop of water surface in direction of flow per running foot of channel, in feet. (If water surface rises in direction of flow, s is negative).

f = lost head per running foot of channel due to friction between the flowing water and the channel perimeter, in feet. ($f = n\,Q^2$, as demonstrated below, and is always positive).

For general treatment, these quantities are to be regarded as coincident factors at one and the same cross-section of the stream.

Let $\int_0^l i\,d\,l$ = total invert drop between two sections l ft. apart $(= I)(= i\,l$, if i is constant), in feet.

$\int_0^l s\,d\,l$ = total water-surface drop between two sections l ft. apart $(= S)$ $(= s\,l$, if s is constant), in feet.

$\int_0^l f\,d\,l$ = total lost head due to perimeter friction between two sections l ft. apart $(= f\,l$, if f is constant), in feet.

F' = total lost head due to internal friction between two sections l ft. apart (eddies, impact, sudden velocity changes, etc.), in feet.

F = total lost head from all sources between two sections l ft. apart $(= F' + \int_0^l f\,d\,l)$, in feet.

F and F' are always positive, while I and S are positive if their gradients drop in the direction of flow, but negative if they rise in that direction.

Evaluation of the Friction Factor, f

The friction factor, f, at any particular section may be evaluated as follows: f is the rate of head loss per foot of channel due to perimeter friction. Hence, $W\,f$ equals the energy in foot-pounds lost, or annihilated, as W moves forward 1 ft., $W = w\,Q = w\,a\,v$ and, therefore, the lost energy $= w\,a\,v\,f$ (Criterion (1)).

This energy loss may be analyzed as follows: $Q = a\,v$ and, hence, v is the length of channel occupied by the quantity of water, Q, or W; and the area of contact between W and the interior of the channel is $p\,v$.

Experiments have demonstrated that the unit frictional resistance is approximately proportional to the square of the mean velocity; that is, the unit frictional resistance, in pounds per square foot of water contact, acting in a direction opposite to that of the stream flow, is Kv^2, K being a constant.

Hence, the total frictional resistance acting against the flow of the quantity of water, W, is $(p\,v) \times (K\,v^2) = K\,p\,v^3$ lb.; and, the energy, in foot-pounds consumed in overcoming this resistance as W moves forward 1 ft., is $K\,p\,v^3$ (Criterion (2)).

From Criteria (1) and (2):

$$K\,p\,v^3 = w\,a\,v\,f,\ v^2 = \frac{w}{K}\,\frac{a}{p}\,f, \text{ and } v = C\sqrt{r\,f} \ldots\ldots\ldots\ldots (1)$$

Equation (1) is the Chezy formula in general terms, in which, f is the friction factor, $r = \frac{a}{p}$, and $C = \sqrt{\frac{w}{K}}$ = the Chezy coefficient.

$$K = \frac{w}{C^2} = \frac{62.5}{C^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

It is to be noted that the Chezy formula regards the flow as being governed wholly by the perimeter friction.

From Equation (1),

$$f = \frac{p}{a}\,\frac{v^2}{C^2} = \frac{p}{a^3\,C^2}\,Q^2 = n\,Q^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Since $h = m\,Q^2$ and $f = n\,Q^2$, then,

$$f = \frac{n}{m}\,h \quad \ldots\ldots (4)$$

$$h = \frac{m}{n}\,f\left(\frac{m}{n} = \frac{1}{2\,g\,a^2}\,\frac{a^3\,C^2}{p} = \frac{a\,C^2}{2\,p\,g} = \frac{C^2}{2\,g}\,r\right) \quad \ldots\ldots (5)$$

Steady Non-Uniform Flow

Fig. 1 represents a general case of "steady flow" in an open channel in which the velocity, v, is variable and the longitudinal slopes of the water surface and bed of channel are not parallel. $x\,x$ and $y\,y$ are any two cross-sections l ft. apart, y being down stream from x. The assumption of "steady flow" means that the same quantity of water per second is passing every cross-section of the channel; that is, $Q_x = Q_y$ and $W_x = W_y$.

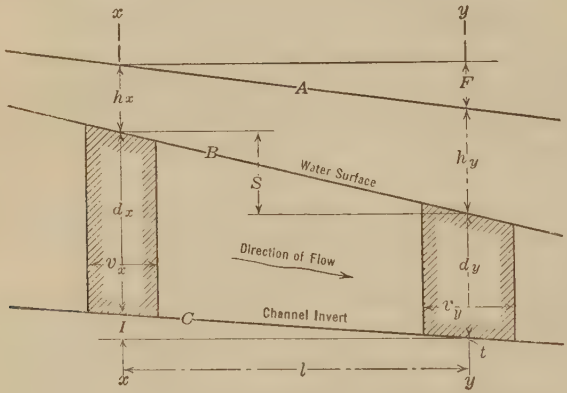


FIG. 1.—LONGITUDINAL SECTION OF FLOWING STREAM.

The quantity of water, W, occupies the position indicated by the hatched area at Section $x\ x$; and in flowing from x to y, it moves to the position shown by the hatched area at Section $y\ y$. The total energy possessed by this quantity of water, W, with respect to a common datum through the low point, t, is less at y than at x by the amount of energy lost in friction between x and y. Since F is the total head loss, $W\,F$ equals the lost energy.

The static heads with respect to Point t are $(d_x + I)$ at x and d_y at y, and hence the potential energies with respect to t are $W\,(d_x + I)$ at x and $W\,d_y$ at y. The kinetic energy at $x = \frac{W\,v_x^2}{2\,g} = W\,h_x$. The kinetic energy at $y = \frac{W\,v_y^2}{2\,g} = W\,h_y$.

Hence,

$$W\,(d_x + I) + W\,h_x = W\,d_y + W\,h_y + W\,F$$

or,

$$d_x + h_x + I = d_y + h_y + F \quad \ldots\ldots (6)$$

This relation is shown graphically in Fig. 1, from which it is also apparent that,

$$d_x + I = d_y + S \text{ and } h_x + S = h_y + F \dots\dots\dots\dots (7)$$

The quantity, $(d + h)$, is a measure of the energy available at any section and Lines (A), (B), and (C) of Fig. 1 may be designated respectively, "energy gradient", "hydraulic gradient", and "invert gradient". Line (A) always drops in the direction of flow. Lines (B) and (C) usually do the same but, under certain local conditions, either or both of them may rise in the direction of flow.

As a general case, Lines (A), (B), and (C), would not be straight lines. They may, however, be considered as straight for a short elemental length, $d\,l$; and in that length, $I = i\,d\,l$, $S = s\,d\,l$, and $F = f\,d\,l$ (making the Chezy formula assumption that the perimeter friction is the only source of loss).

Then, from Equations (6) and (7):

$$(d_x + h_x) - (d_y + h_y) = d\,l\,(f - i) \dots\dots\dots\dots (8)$$

$$h_x - h_y = d\,l\,(f - s) \dots\dots\dots\dots (9)$$

From Equation (1), $v = C\sqrt{r\,f}$. Hence the formula, $v = C\sqrt{r\,s}$, can be true only if $f = s$, and from Equation (9), this can be true only if h (and, hence, v) is constant. Likewise, if $f = i$, then $v = C\sqrt{r\,i}$, and from Equation (8), it is seen that in this case $(d + h)$ is constant.

In Equation (6), group the unknown quantities in the left-hand member and the known quantities on the right, as follows:

$$(d_x + h_x) = (d_y + h_y) + (F - I) \dots\dots\dots\dots (10)$$

if proceeding up stream, or

$$(d_y + h_y) = (d_x + h_x) + (I - F) \dots\dots\dots\dots (11)$$

if proceeding down stream.

Equations (10) and (11) are applicable to any invert profile and to any shape of channel section, constant or variable, open or closed, flowing freely or under pressure. The stretches, l, between Sections x and y should be taken sufficiently short to produce the desired degree of accuracy and should terminate at all sudden changes in channel section or invert slope. In case of a sudden head loss, sudden change of section, or a vertical rise or drop in invert elevation, consecutive values of $(d + h)$ may be computed by taking l as zero.

From Equations (10) and (11), the energy gradient and the hydraulic gradient (Lines (A) and (B) of Fig. 1) can be computed for any given channel for an assumed Q.

In proceeding up stream, $(d_x + h_x)$, as computed from Equation (10), becomes $(d_y + h_y)$ for the next up-stream stretch of length, l; and in proceeding down stream, $(d_y + h_y)$, as computed from Equation (11), becomes $(d_x + h_x)$ for the next down-stream stretch.

In order to start this procedure, however, a section must be found in which d and h are known for the assumed Q, and the only place where these quantities are independently known in the case of non-uniform flow is at the "critical section" discussed later under that heading.

Equations (10) and (11) cannot be solved directly for the unknown quantities in the left-hand members. In the first place the term, F, in the right-hand members can be known only approximately until the left-hand members are evaluated. Furthermore, at every section except the "critical section" there will be two possible mathematical values of both d and h for any particular value of $(d+h)$ for a given Q, one combination of d and h giving a small stream section flowing at a high velocity, and the other a larger stream section flowing at a slower velocity.

This may be graphically illustrated as follows: Assume for simplicity a rectangular channel, 10 ft. wide. The coincident values of d and $(d + h)$ for flows of 100 and 200 sec-ft. are computed in Table 1 and plotted in Fig. 2. Assume a Q of 100 sec-ft. in this channel and $(d + h) = 3$. Then, from Fig. 2, the two possible values of d are seen to be 0.84 and 2.80.

TABLE 1.—Q-CURVES FOR RECTANGULAR SECTION OF FIG. 2.

d.	$a = 10\,d$.	$m = \frac{1}{2\,g\,a^2}$.	$Q = 100$ sec-ft.		$Q = 200$ sec-ft.	
			$h = m\,Q^2$	$d + h$.	$h = m\,Q^2$	$d + h$.
6.0	60	0.0000043	0.04	6.04	0.17	6.17
4.0	40	0.0000097	0.10	4.10	0.39	4.39
3.0	30	0.0000173	0.17	3.17	0.69	3.69
2.0	20	0.0000389	0.39	2.39	1.55	3.55
1.0	10	0.0001555	1.55	2.55	6.22	7.22
0.5	5	0.0006219	6.22	6.72	24.88	25.38

The general criterion for selecting the correct value of d as between these two mathematical possibilities is as follows: The water will flow at such a depth, d, that the lost head, F, between any two sections will be as nearly as possible equal to the invert drop, I, in that same stretch of channel. Referring to Equations (10) and (11), it is seen that this criterion may be expressed as follows: Successive values of $(d + h)$ will be as nearly as possible equal to each other (Criterion (3)). The application of this criterion is more fully discussed elsewhere in this paper.

The computation of the hydraulic gradient is thus a matter of locating the "critical section" and, with that as a starting point, applying Equation (10) or (11) as guided by Criterion (3).

THE CRITICAL SECTION

The "critical section" of a flowing stream is the "neck of the bottle" and controls the flow of the entire stream. The "critical section" is delivering the maximum, Q, of which it is capable for the particular value of $(d + h)$ under which it is operating. In other words, the "critical section" is flowing at "maximum efficiency" and is, in general, the only section doing so.

It is apparent from Fig. 2 that for any particular value of $(d + h)$ in a known section, as d increases from zero to $(d + h)$, h decreases from $(d + h)$ to zero and Q increases from zero to a certain maximum and again decreases to zero. For instance, it is seen that in a rectangular section, 10 ft. wide, 200 sec-ft. is the maximum flow that can be delivered with $(d + h)$ at a value of 3.48; or, conversely, 3.48 is the minimum value of $(d + h)$ which will deliver 200 sec-ft. It is also to be noted that for a $(d + h)$ of 3.48 the depth of flow, d, which will deliver the maximum, Q, is 2.32. Therefore, 2.32 is the "critical depth" for a $(d + h)$ of 3.48.

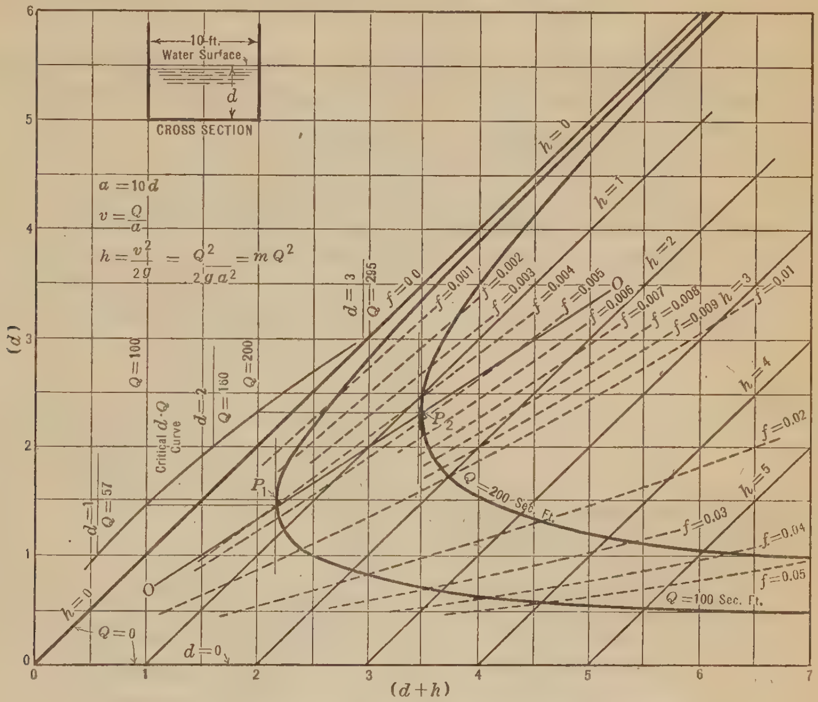


FIG. 2.—ANALYSIS OF RECTANGULAR CHANNEL SECTION.

Hence, for a flow of 200 sec-ft. $(d + h) = 3.48$, $d = 2.32$, and $h = 1.16$, if the section is acting as a "critical section". Otherwise, $(d + h)$ will be greater than 3.48 and d and h will each have two possible values as shown by the 200-curve of Fig. 2, the correct values to be determined by Criterion (3).

A direct method for evaluating d and h, and likewise f, at the "critical section" may be derived in general terms by the aid of calculus, as follows: The value of h to produce a maximum, Q, for a given $(d + h)$ is determined by placing $\frac{d\,Q}{d\,h} = 0$.

Since $Q = a\,v$,

$$d\,Q = a\,(d\,v) + v\,(d\,a) \qquad (12)$$

Since $v = \sqrt{2\,g\,h}$,

$$d\,v = \frac{\sqrt{2\,g}}{2\,\sqrt{h}}\,d\,h \qquad (13)$$

(assuming uniform velocity throughout the area, a)

From Equations (12) and (13),

$$d\,Q = \frac{a\sqrt{2\,g}}{2\sqrt{h}}\,d\,h + \sqrt{2\,g\,h}\,d\,a$$

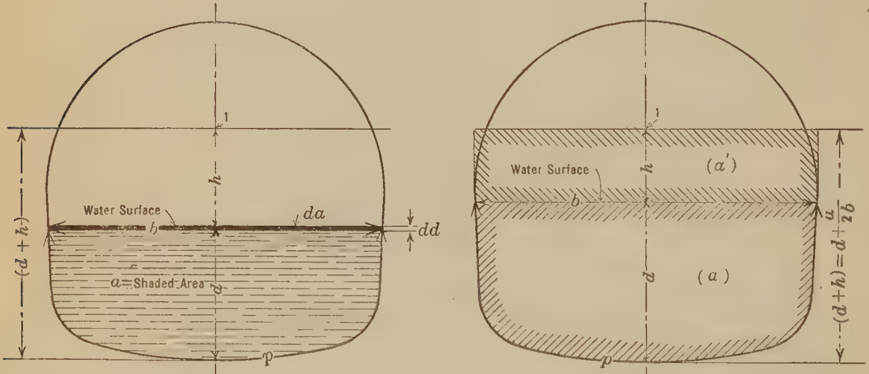
and for maximum, Q,

$$\frac{d\,Q}{d\,h} = \frac{a\sqrt{2\,g}}{2\sqrt{h}} + \sqrt{2\,g\,h}\,\frac{d\,a}{d\,h} = 0 \,\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Fig. 3(a) represents a general case of an irregular section, from which it is seen that $d\,a = b\,d\,d = -b\,d\,h$ for a given value of $(d + h)$.

Hence, $\frac{d\,a}{d\,h} = -b$, and from Equation (14),

$$\frac{a\sqrt{2\,g}}{2\sqrt{h}} - b\sqrt{2\,g\,h} = 0,\ \frac{a}{2\sqrt{h}} = b\sqrt{h},\text{ and } a = 2\,b\,h \ldots\ldots (15)$$



$h =$ Velocity head $= \frac{v^2}{2g} = \frac{Q^2}{2ga^2} = m\,Q^2$

$f =$ Longitudinal slope of point $(1) = \frac{pQ^2}{a^3C^2} = n\,Q^2$

For a definite fixed Value of $(d+h)$ an increase in d means an equal Decrease in h and hence $dd = -dh$

(a) GENERAL CASE

Area $(a) = 2\,bh = 2\,(a')$

$\frac{a^3}{b} = \frac{Q^2}{g} = 0.0311\,Q^2$

$h =$ Velocity head $= \frac{a}{2b}$

$f =$ Longitudinal slope of point $(1) = \frac{pg}{bC^2}$

(b) CRITICAL SECTION

Fig. 3.—Cross-Section of Flowing Stream.

This relation for a maximum, Q, is shown graphically in Fig. 3(b), in which the area, a, is double the area, a'. Hence, at the critical section, from Equation (15):

$$h = \frac{a}{2\,b} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

and as $Q = a\,v = a\sqrt{2\,g\,h}$, at the critical section:

$$Q^2 = a^2\,2\,g\,h = \frac{a^3\,g}{b} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

$$\frac{a^3}{b} = \frac{Q^2}{g} = 0.0311\,Q^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

$$Q = 5.671 \sqrt{\frac{a^3}{b}} \text{ .... } (19)$$

Also,

$$f = n\ Q^2 = \frac{p\ Q^2}{a^3\ C^2} = \frac{p}{a^3\ C^2} \frac{a^3\ g}{b}$$

Hence, at the critical section,

$$f = \frac{p\ g}{b\ C^2} \text{ .... } (20)$$

The three pertinent factors, *d, h,* and *f*, at the critical section may now be computed as follows: *d* is obtained by trial or by direct analysis from Equation (18), which also furnishes the corresponding coincident values of *a, b,* and *p*; *h* is computed from Equation (16); and *f* is computed from Equation (20). Knowing *d, h,* and *f* at the critical section, the values of these factors at other points of the stream are computed progressively from Equation (10) or Equation (11).

If the rectangular section of Fig. 2 is acting as a critical section and $Q = 100$, from Equation (18),

$$\frac{a^3}{b} = 311$$

In this case,

$$\frac{a^3}{b} = b^2\ d^3 = 100\ d^3$$

hence,

$$d^3 = 3.11$$

and,

$$d = 1.46$$

From Equation (16),

$$h = \frac{a}{2\ b} = \frac{d}{2} = 0.73$$

$$d + h = 2.19$$

and,

$$p = b + 2\ d = 10 + 2.92 = 12.92$$

From Equation (20),

$$f = \frac{12.92 \times 32.16}{10\ C^2}$$

and, if $C = 100$, $f = 0.0042$.

For $Q = 200$, $d^3 = 12.44$; $d = 2.32$; $h = 1.16$; $d + h = 3.48$; and $p = 14.64$; therefore,

$$f = \frac{14.64 \times 32.16}{10 \times 10\ 000} = 0.0047$$

The "critical values" of *d* and $(d + h)$ are located at Points P_1 and P_2 of Fig. 2 and, at these points, the *Q*-curve tangents must be vertical.

For an irregular section the procedure outlined may become rather complicated. In such a case the critical points, or Q-curve vertices, may be located as follows for the rectangular section of Fig. 2:

1.—Compute a locus of these vertices as determined by the coincident values of (d) and $(d + h)$ of Table 2.

TABLE 2.—CRITICAL FACTORS FOR RECTANGULAR SECTION OF FIG. 2.

d.	a.	b.	h.	$d + h$.	Q.
			From Equation (16).		From Equation (19).
3	30	10	1.5	4.5	295
2	20	10	1.0	3.0	160
1	10	10	0.5	1.5	57

2.—This locus plots as shown by Line OO of Fig. 2. For a rectangular section it is a straight line, but for an irregular section the locus, in general, would be curved.

3.—Plot a critical d Q-curve from the (d) and (Q)-columns of Table 2 as indicated at the left of Fig. 2.

4.—The intersection of the proper critical (d) line with the locus, OO, will locate the vertex of any desired Q-curve.

It is also a simple matter to plot a set of f-curves crossing the Q-curves of Fig. 2. The f-curves are computed from Equation (5) as illustrated in Table 3 for the rectangular section of Fig. 2.

TABLE 3.—f-CURVES FOR RECTANGULAR SECTION OF FIG. 2.*

d.	a.	p.	$m = \frac{1}{2\,g\,a^2}$.	$n = \frac{p}{a^3\,C^2}$.	$\frac{m}{n}$.	h.
						From Equation (5).
3.0	30	16	0.0000173	0.000000059	293.22	293.22 f.
2.0	20	14	0.0000389	0.000000175	222.29	222.29 f.
1.0	10	12	0.0001555	0.000001200	129.58	129.58 f.
0.5	5	11	0.0006219	0.000008800	70.67	70.67 f.

* C is given a constant value of 100.

It is apparent from Table 3 that for any particular value of d, h varies directly as f and, hence, the $(d + h)$ increments will vary directly as the f increments on the same d line. In other words, equal distances on any horizontal line of Fig. 2 represent equal increments of f.

Hence, the f-curves are easily plotted by graduating the horizontal d lines as follows: For $d = 3$ in Table 3, if $f = 0$, $h = 0$, and $(d + h) = 3.0$; if $f = 0.01$, $h = 2.93$, $(d + h) = 5.93$, and that portion of the $d = 3$ line, between $(d + h) = 3$ and $(d + h) = 5.93$, may be divided into ten equal parts to mark f values of 0.001, 0.002, 0.003, etc. For $d = 1$, if $f = 0.05$,

$h = 6.48$, and $(d + h) = 7.48$; and by dividing that portion of the $d = 1$ line, between $(d + h) = 1$ and $(d + h) = 7.48$, into five equal parts, these divisions will be points on the f-curves, 0.01, 0.02, 0.03, 0.04, 0.05, etc.

The dotted lines of Fig. 2 are plotted in this manner and are seen to check the computed values of $f = 0.0042$ and $f = 0.0047$ at Points P_1 and P_2, respectively.

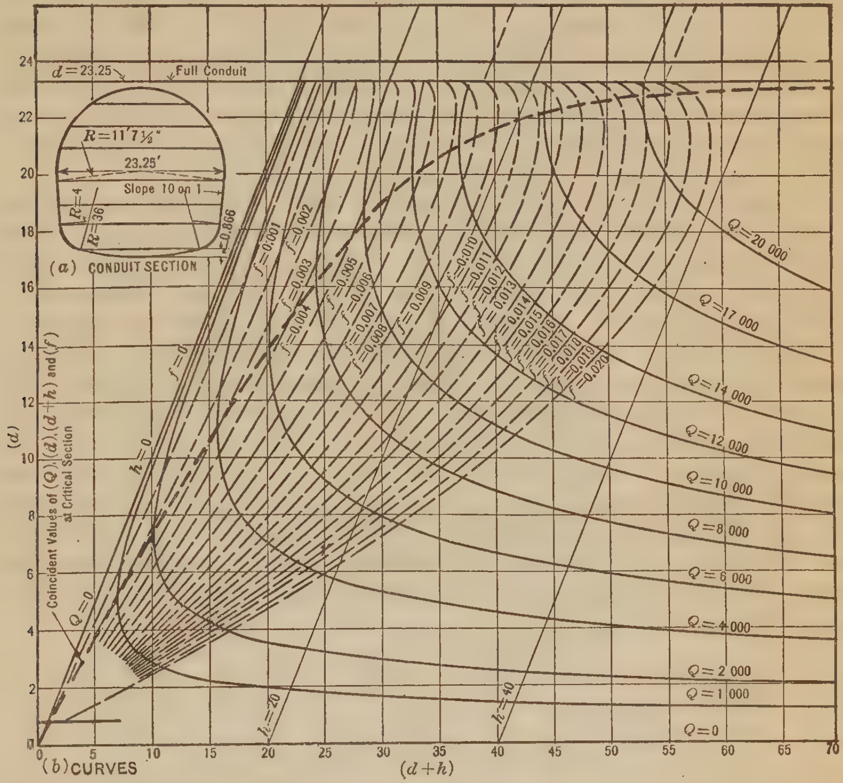


FIG. 4.—ANALYSIS OF CONDUIT.

Fig. 4(b) is a chart embodying the principles of Fig. 2 as applied to the conduit of Fig. 4(a). The preparatory computations for Fig. 4(b) are indicated in Table 4 in which the assumed values of C are based on the hydraulic radius, r. The heights from the bottom to the horizontal lines (Fig. 4(a)), are given under d. The dotted critical factor line of Fig 4(b) becomes horizontal at infinity, and for any finite value of h the "critical depth" is less than the maximum vertical depth of the conduit. It is also apparent that when the conduit is flowing full or under pressure, a change in (d) does not alter the area, a, and, consequently, (h) and (f) remain constant for a given Q, and Fig. 4(b) may be extended to include pressure flows by straight line extensions of the various (f) and (Q)-curves above the points where they intersect the ($d = 23.25$)-line. All these extensions would be sloped parallel to the (h)-lines of Fig. 4(b).

The computations indicated in Table 4 are, of course, much simplified for a geometrical channel section in which b, a, and p can be expressed as simple algebraic functions of the depth, d.

Location of Critical Section.—The first question which comes up in the application of the foregoing principles is, Where is the "critical section" located? As a general proposition, the channel section may possibly have an entirely different shape at one point than at another and hence the "critical section" may be at different locations for different values of Q, due to relative changes in the hydraulic elements of the various sections at different depths of flow.

The only general criterion for the location of the critical section for a given Q comes from the fact that the value of $(d + h)$ at any section as computed from Equations (10) or (11) cannot be less than the "critical value" of $(d + h)$ for that section for the assumed Q.

Hence, the "critical section" will be that section from which one can start with the critical values of d and h for that section and, by applying progressively Equation (10) or Equation (11), or both, as the case may be, obtain, at all other sections of the stream, values of $(d + h)$ none of which will be less than the critical $(d + h)$ of those respective sections for the assumed Q.

In a channel of varying cross-section, it becomes a matter of selecting a section which judgment would dictate as the probable "critical section". The correctness or incorrectness of this guess will be proven by the progressive application of Equation (10) or Equation (11). If, at any stage of this procedure, a section is encountered at which $(d + h)$, as computed from Equation (10) or Equation (11), is less than the critical $(d + h)$ of that section as computed from Equations (18) and (16), then that section is established as the "critical section" as regards all sections previously computed and a new start is made at that section, the previous computations being revised in reverse order.

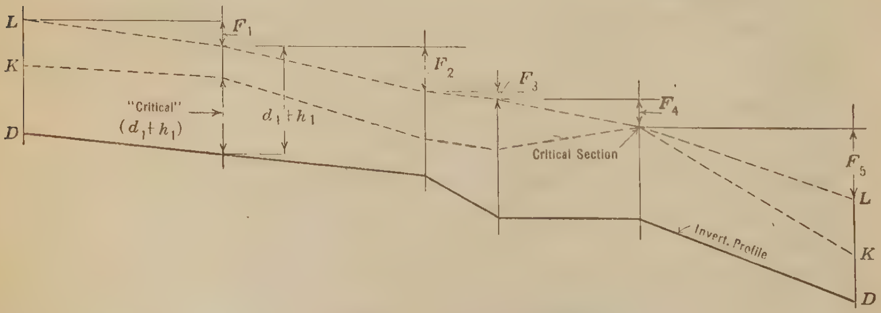


FIG. 5.—CRITICAL SECTION CRITERION.

These principles are graphically represented in Fig. 5, in which, Line $D\,D$ is the invert profile; Line $K\,K$ is a locus of critical values of $(d + h)$; that is, the vertical intercept between the D-line and the K-line at any section is the critical value of $(d + h)$ at that section for the assumed Q; and Line $L\,L$ is the "energy gradient" and its successive drops represent the friction losses, F, in the various stretches of the channel. Hence, Line $L\,L$ cannot drop below

TABLE 4.—HYDRAULIC ELEMENTS OF CONDUIT OF FIG. 4.

d.	b.	a.	p.	$r = \frac{a}{p}$.	C.	$m^* = \frac{1}{2\,g\,a^2}$.	$n^\dagger = \frac{p}{a^3\,C^2}$.	$\frac{m}{n}$.	CRITICAL FACTORS. (See Dotted Line of Fig. 4(b).)					
						Prefix 0.00000.	Prefix 0.0000000.		$h = \frac{a}{2\,b}$.	$Q^2 = \frac{h}{m}$.	Q.	$v = \frac{Q}{a}$.	$d + h$.	$f = n\,Q^2$.
0.866	15.70	9.10	15.82	0.58	95	......	......	80.7	0.29	1 521	39	4.30	1.16	0.0035
4.372	21.92	79.87	25.84	3.09	122	24372	03407	715.3	1.82	748 225	865	10.80	6.19	0.0025
7.000	22.44	138.16	31.12	4.44	127	08144	00732	1 112.6	3.08	3 779 136	1 944	14.10	10.08	0.0028
10.469	23.13	217.20	38.10	5.70	131	03296	00217	1 518.8	4.70	14 250 625	3 775	17.40	15.17	0.0031
11.625	23.25	244.10	40.42	6.04	131	02609	00162	1 610.5	5.25	20 115 225	4 485	18.40	16.87	0.0033
15.000	22.25	321.31	47.27	6.80	132	01506	00082	1 836.6	7.22	47 941 776	6 924	21.55	22.22	0.0039
18.000	19.44	384.52	53.92	7.13	133	01052	00054	1 948.1	9.89	94 051 204	9 698	25.20	27.89	0.0051
21.000	13.75	435.34	62.23	7.00	133	00820	00043	1 907.0	15.83	192 904 321	13 889	31.90	36.83	0.0083
23.000	4.80	455.56	72.11	6.32	132	00749	00044	1 702.3	47.45	633 428 224	25 168	55.25	70.45	0.0279
23.250	0	456.34	76.94	5.93	131	00746	00047	1 587.2	∞	∞	∞	∞	∞	∞

[*] $m_1 = 0.00018774$.

[†] $n_1 = 0.000002326$.

Line $K\ K$ at any point and will be tangent to Line $K\ K$ at the "critical section", as indicated in Fig. 5.

The only reason the "critical section" cannot be definitely located by this method is that the F-drops are not known until the various water depths are evaluated. These principles may be applied as an aid to judgment, however, by computing tentative F-values and plotting a trial $L\ L$-line and shifting it vertically with its datum kept horizontal in order to locate a point of tangency with Line $K\ K$ as a first guess on the critical section location.

In many instances, the proposed tentative F-values might logically be based on the supposition that each and every section is acting as a "critical section". The matter of locating the critical section is, of course, much simplified if the channel cross-section is constant, as in the following example.

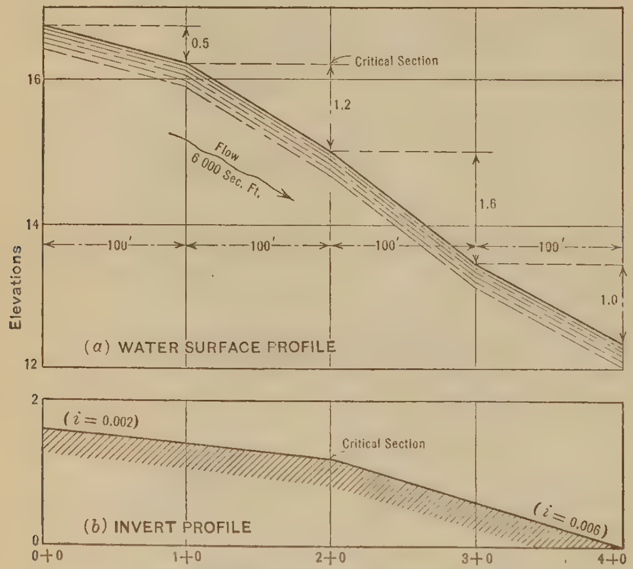


FIG. 6.—ILLUSTRATIVE EXAMPLE.

Illustrative Example.—Let Fig. 4(a) represent a conduit of constant cross-section to be analyzed for a flow of 6 000 sec-ft. (see Fig. 6 and Table 5). Suppose the invert slope, i, is constant at 0.002 in the up-stream portion of the conduit, changing abruptly to 0.006 at some point between the inlet and the outlet. In a short elemental length, $d\ l$, assume that $f_x = f_y$ and, considering perimeter friction as the only source of loss, there results from Equation (8),

$$(d_x + h_x) = (d_y + h_y) + d\ l\ (f - i)$$

from which it is evident that $(d + h)$ is increasing down stream if $i > f$, and is decreasing down stream if $i < f$. Also, if the channel section is constant, $(d + h)$ will be less at the "critical section" than at any other section of the stream. Hence, in a channel of constant section, if the critical section is located in a stretch where i is greater than the critical f, it must be at the up-stream end of that stretch; and if in a stretch where i is less than the critical f, it will be at the down-stream end.

From Fig. 4(b), $f = 0.0037$ at the critical section in the present example ($Q = 6\,000$), and, hence, the critical section must be at the up-stream end of the 0.006 slope, or at the down-stream end of the 0.002 slope. Since these two points coincide, the critical section is definitely located. If the slopes had been reversed, however, with 0.006 at the up-stream end and 0.002 at the down-stream end, there would have been two possible locations of the critical section to investigate, one at the inlet and one at the outlet, with perhaps the possibility of a critical section at each end and a hydraulic jump within the conduit to bridge the change from low stage to high stage.

TABLE 5.—ANALYSIS OF CONDUIT WITH CONSTANT CROSS-SECTION.*

Station.	$d + h$.	d.	h.	f.	F.	I.	S.	Invert elevation.	Water surface elevation.
0 + 0	20.49	15.3	5.39	0.0029				1.6	16.7
					0.30	0.2	0.5		
1 + 0	20.39	14.8	5.59	0.0031				1.4	16.2
					0.34	0.2	1.2		
2 + 0	20.25	13.8	6.45	0.0037				1.2	15.0
					0.41	0.6	1.6		
3 + 0	20.44	12.8	7.64	0.0045				0.6	13.4
					0.47	0.6	1.0		
4 + 0	20.57	12.4	8.17	0.0049				0	12.4

* See Fig. 6.

The hydraulic gradient may now be computed by starting at the critical section which, in this case, is between the inlet and the outlet, and proceeding from there both up stream and down stream. Hence, start at the vertex of the $Q = 6\,000$-curve and apply Equation (10) to go up stream, or Equation (11) to go down stream.

Let $I = i\,l$ and assume $F = \left(\frac{f_x + f_y}{2}\right) l$, taking l in 100-ft. stretches. To satisfy Criterion (3), F must be as nearly as possible equal to I or, in the present case, $\frac{1}{2}\,(f_x + f_y)$ must be, as nearly as possible, equal to i.

In proceeding up stream from the critical section, $f_y = 0.0037$ and $i = 0.002$, and it is apparent that one must follow the Q-curve upward from the dotted line of Fig. 4(b) to make $\frac{1}{2}\,(f_x + f_y)$ approach the value of i.

Hence, there is the following general criterion in the application of Fig. 4(b), or a similar chart for any constant-section channel, providing f is the only source of head loss: Follow the Q-curve upward or downward from its vertex accordingly as f at the vertex is, respectively, greater or less than i (Criterion (4)).

In this example there is, at the critical section, at the break in invert gradients, $d = 13.8$, $(d + h) = 20.25$, and $f = 0.0037$, as read from the vertex of the $Q = 6\,000$-curve of Fig. 4(b).

Proceeding up stream and following the Q-curve upward from its vertex, per Criterion (4), it is found that, at a point 100 ft. up stream, $(d + h) = 20.39$ and $f = 0.0031$, to satisfy Equation (10) in the following form:

$$(d_x + h_x) - \frac{l}{2} f_x = (d_y + h_y) + \frac{l}{2} f_y - l i \ldots\ldots\ldots\ldots (21)$$

$\left(\text{since } F = \left(\frac{f_x + f_y}{2}\right) l \text{ and } I = i\, l\right)$.

From Equation (21),

$$(d_x + h_x) - 50 f_x = 20.25 + 0.185 - 0.2 = 20.235$$

and,

$$20.39 - (50 \times 0.0031) = 20.235$$

The corresponding water depth, d, as given by Fig. 4(b), is 14.8.

At the up-stream end of the next 100-ft. stretch $(d + h) = 20.49$ and $f = 0.0029$ as follows:

$$(d_x + h_x) - 50 f_x = 20.39 + (50 \times 0.0031) - 0.2 = 20.345$$
$$20.49 - (50 \times 0.0029) = 20.345$$
$$d = 15.1$$

and so on to the inlet.

Proceeding down stream from the critical section, $i = 0.006$ and, therefore, it must follow the Q-curve downward from its vertex. At a point 100 ft. down stream from the critical section, $(d + h) = 20.44$ and $f = 0.0045$ from the following adaptation of Equation (11):

$$(d_y + h_y) + \frac{l}{2} f_y = (d_x + h_x) - \frac{l}{2} f_x + l i \ldots\ldots\ldots\ldots (22)$$

$$(d_y + h_y) + 50 f_y = 20.25 - 0.185 + 0.6 = 20.665$$
$$20.44 + (50 \times 0.0045) = 20.665$$
$$d = 12.8$$

At the down-stream end of the next 100-ft. stretch, $(d + h) = 20.57$ and $f = 0.0049$ as follows:

$$(d_y + h_y) + 50 f_y = 20.44 - (50 \times 0.0045) + 0.6 = 20.815$$
$$20.57 + (50 \times 0.0049) = 20.815$$
$$d = 12.4$$

and so on to the outlet.

These results are given in Table 5 and plotted in Fig. 6. In this example, d was computed at points a definite distance apart. This is the method that would be used in the precise analysis of a channel of changing cross-section, plotting curves similar to those of Fig. 4(b) for each section considered.

In a channel of constant cross-section this process can be simplified somewhat by assuming definite values of d and then computing where these depths would occur by solving directly for l instead of assuming l and solving for d by trial and error.

For instance, starting with $d = 13.8$ $(d + h) = 20.25$, and $f = 0.0037$ at the critical section, it is possible to determine directly where $d = 14.8$ as follows: If $d_x = 14.8$, then from Fig. 4(*b*), $(d_x + h_x) = 20.39$ and $f_x = 0.0031$. Then from Equation (21),

$$l\left(\frac{f_x}{2} + \frac{f_y}{2} - i\right) = (d_x + h_x) - (d_y + h_y)$$

$$l\,(0.00155 + 0.00185 - 0.002) = 20.39 - 20.25$$

$$l = \frac{0.14}{0.0014} = 100$$

If the water level is known at one or more points of the channel, the discharge will be a value of Q which will satisfy the known conditions without resulting in a value of $(d + h)$ at any section less than its critical $(d + h)$ for that Q.

In such a case a situation might be found in which no section of the channel would be operating as a "critical section". If the velocity of approach above the conduit inlet and the velocity of the tail-water below its outlet are so small that they may be neglected, then h at those points becomes zero and Equation (10) becomes,

$$d_x = d_y + F - I, \text{ or } F = d_x - d_y + I$$

in which, d_x and d_y are in the still water of the approach pool and tail-water pool, respectively; I is the total invert drop between those points; and F is the total head loss from all sources.

In this case the discharge would be the maximum value of Q which could be delivered with a total loss of head equal to this value of F, unless the resulting $(d + h)$ at some section should be below its critical value, in which case the discharge would be governed by the "critical section". The analysis, of course, must include the F losses at inlet and outlet or elsewhere due to sudden velocity changes.

If the channel in the foregoing example had not been of constant cross-section it would have been necessary to prepare a chart similar to Fig. 4(*b*) for each of the different channel sections, applying each one in its proper place and observing the general criterion in the location of the "critical section". A sudden change of section would introduce a sudden head loss and a corresponding sudden drop in the value of $(d + h)$.

WEIRS, DAMS, AND FALLS

The conditions at the crest of a weir, dam, or falls are usually those of a critical section; that is, the stream section on the crest is delivering the maximum Q possible for the particular value of $(d + h)$ under which it is operating, without hindrance or limitation from other sections up stream or down stream.

Assuming the stream section on the crest to be rectangular, $a = b\,d$ and the following simplifications of the general critical section Equations (16) and (19) occur, representing conditions at the weir crest.

From Equation (16),

$$h = \frac{d}{2} \quad \text{.................................} (23)$$

and,

$$d + h = 3\ h = \frac{3}{2}\ d \dots\dots\dots\dots\dots\dots\dots\dots (24)$$

From Equation (19),

$$Q = 5.671\ b\ d^{\frac{3}{2}} \dots\dots\dots\dots\dots\dots\dots\dots (25)$$

and,

$$d = 0.3145 \left(\frac{Q}{b}\right)^{\frac{2}{3}} \dots\dots\dots\dots\dots\dots\dots\dots (26)$$

Equation (25) should give the discharge over any form of rectangular weir, dam, or falls, b and d being the exact dimensions of the stream of water on the weir crest.

Because of difficulties in the way of measuring d at the crest, weir discharge is usually expressed in terms of the static head a "short distance up stream" from the weir. This head, usually designated as H, is the difference in elevation between the weir crest and the water surface at the section where H is measured, which corresponds to Section x of Fig 1, and the weir crest corresponds to Section y.

Neglecting friction between x and y, Equation (10), as shown in Fig. 7, becomes,

$$d_x + h_x + I = d_y + h_y \dots\dots\dots\dots\dots\dots\dots\dots (27)$$

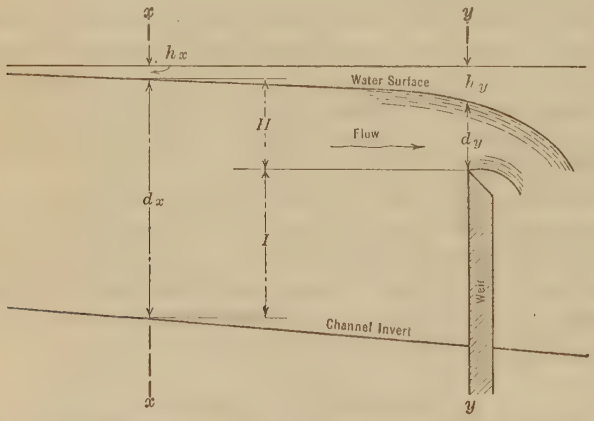


Fig. 7.—Longitudinal Section of Flowing Stream Above a Weir.

In Fig. 7, I is negative since the invert is higher at y than at x. Also, $d_x + I = H$ and from Equation (24), $d_y + h_y = \frac{3\ d_y}{2}$. Hence, from Equation (27),

$$\frac{3\ d_y}{2} = H + h_x$$

and,

$$d_y = \frac{2}{3}\ (H + h_x) \dots\dots\dots\dots\dots\dots\dots\dots (28)$$

and from Equations (25) and (28),

$$Q = 3.087\ b\ (H + h_x)^{\frac{3}{2}} \qquad (29)$$

In Equation (29), h_x is the effective velocity head at Section x, where H is measured, and b is the net width of the stream at the weir crest.

In the Francis weir formula,

$$Q = K\ b\ H^{\frac{3}{2}} \qquad (30)$$

in which, K is a constant.

From Equations (25) and (30),

$$K\ H^{\frac{3}{2}} = 5.671\ d_y^{\frac{3}{2}}$$

and,

$$K = 5.671 \left(\frac{d_y}{H}\right)^{\frac{3}{2}} \qquad (31)$$

It should be kept in mind that these equations assume a uniform velocity throughout the entire area of every cross-section of the stream. $\left(v = \text{the mean velocity, } \frac{Q}{a}, \text{ and } h = \frac{v^2}{2\ g}.\right)$

If the velocity in any cross-section is not uniform, then $h > \frac{v^2}{2\ g}$. Equation (25) is theoretically correct on the assumption that the velocity on the crest is uniform throughout the area, $b\ d$, b and d being the exact dimensions of the stream on the crest.

Equation (29) is theoretically correct on the further assumption of uniform velocity at Section x and no friction losses between x and y.

In Equation (30), if K is determined experimentally it includes the effect of friction losses and non-uniform velocities, but the evaluation of K from Equation (31) as plotted in Fig. 8, involves the same assumptions as Equation (29).

It is not usually feasible to measure d_y with any degree of accuracy, but the principles of Equation (31) may be analyzed in the following manner:

$$b_y\ d_y = a_y \text{ and } a_y\ v_y = a_x\ v_x$$

from which,

$$b_y\ d_y \sqrt{h_y} = a_x \sqrt{h_x}$$

or,

$$b_y^2\ d_y^2\ h_y = a_x^2\ h_x \qquad (32)$$

From Equation (23), $h_y = \frac{1}{2}\ d_y$, and, from Equation (28), $h_x = \frac{3}{2}\ d_y - H$.

Hence, from Equation (32),

$$\frac{1}{2}\ b_y^2\ d_y^3 = a_x^2 \left(\frac{3}{2}\ d_y - H\right)$$

or,

$$\left(\frac{a_x}{b_y}\right)^2 = \frac{d_y^3}{3\ d_y - 2\ H} \qquad (33)$$

$5.671 = \sqrt{g}$ and, hence, from Equation (31), $d_y = \frac{H K^{\frac{2}{3}}}{g^{\frac{1}{3}}}$. Substituting this value of d_y in Equation (33),

$$\frac{a_x}{b_y} = k H \text{ .... } (34)$$

in which,

$$k = \frac{K}{g^{\frac{1}{3}} \sqrt{3 K^{\frac{2}{3}} - 2 g^{\frac{1}{3}}}} \text{ .... } (35)$$

Coincident values of K and k from Equation (35) and corresponding values of $\frac{a_x}{b_y}$ for various values of H are listed in Table 6 and plotted in Fig. 8.

TABLE 6.—WEIR COEFFICIENTS.

K, in Equation (30).	k' from Equation (37).	k from Equation (35).	$\frac{a_x}{b_y}$ FROM EQUATION (34).*			
			$H = 1.$	$H = 2.$	$H = 3.$	$H = 4.$
3.087†	0	∞	∞	∞	∞	∞
3.1	0.0028	7.2458	7.25	14.49	21.74	28.98
3.15	0.0136	3.3679	3.37	6.74	10.10	13.47
3.2	0.0243	2.5617	2.56	5.12	7.69	10.25
3.3	0.0455	1.9293	1.93	3.86	5.79	7.72
3.4	0.0665	1.6437	1.64	3.29	4.93	6.57
3.5	0.0873	1.4768	1.48	2.95	4.43	5.91
3.6	0.1080	1.3663	1.37	2.73	4.10	5.47
3.7	0.1284	1.2877	1.29	2.58	3.86	5.15
3.8	0.1486	1.2292	1.23	2.46	3.69	4.92
4.0	0.1886	1.1486	1.15	2.30	3.45	4.59
4.2	0.2279	1.0971	1.10	2.19	3.29	4.39
4.5	0.2857	1.0498	1.05	2.10	3.15	4.20
5.671‡	0.5000	1.0000	1.00	2.00	3.00	4.00

* See Fig. 8.

† $3.087 = \sqrt{g} \left(\frac{2}{3}\right)^{\frac{3}{2}}$.

‡ $5.671 = \sqrt{g}$.

The K lines of Fig. 8 would be straight if plotted to natural scales, but logarithmic plotting is used to reduce the size of the diagram. It is apparent from Fig. 7 that H may have any value between the limits, d_y and $(d_y + h_y)$, depending on the location of Section x.

From Equation (31), when $H = d_y$, $K = 5.671$; and when $H = d_y + h_y$ $\left(= \frac{3}{2} d_y\right)$, $K = 5.671 \left(\frac{2}{3}\right)^{\frac{3}{2}} = 3.087$. Hence, K must lie between the two limits, 5.671 and 3.087.

From Equations (35) and (34), for $K = 5.671$ $(= g^{\frac{1}{2}})$, $\frac{a_x}{b_y} = H$; and for $K = 3.087$ $\left(= g^{\frac{1}{2}} \times \left(\frac{2}{3}\right)^{\frac{3}{2}}\right)$, $\frac{a_x}{b_y} = \infty$ for all values of H. That is, $a_x = \infty$, or, in other words, $h_x = 0$.

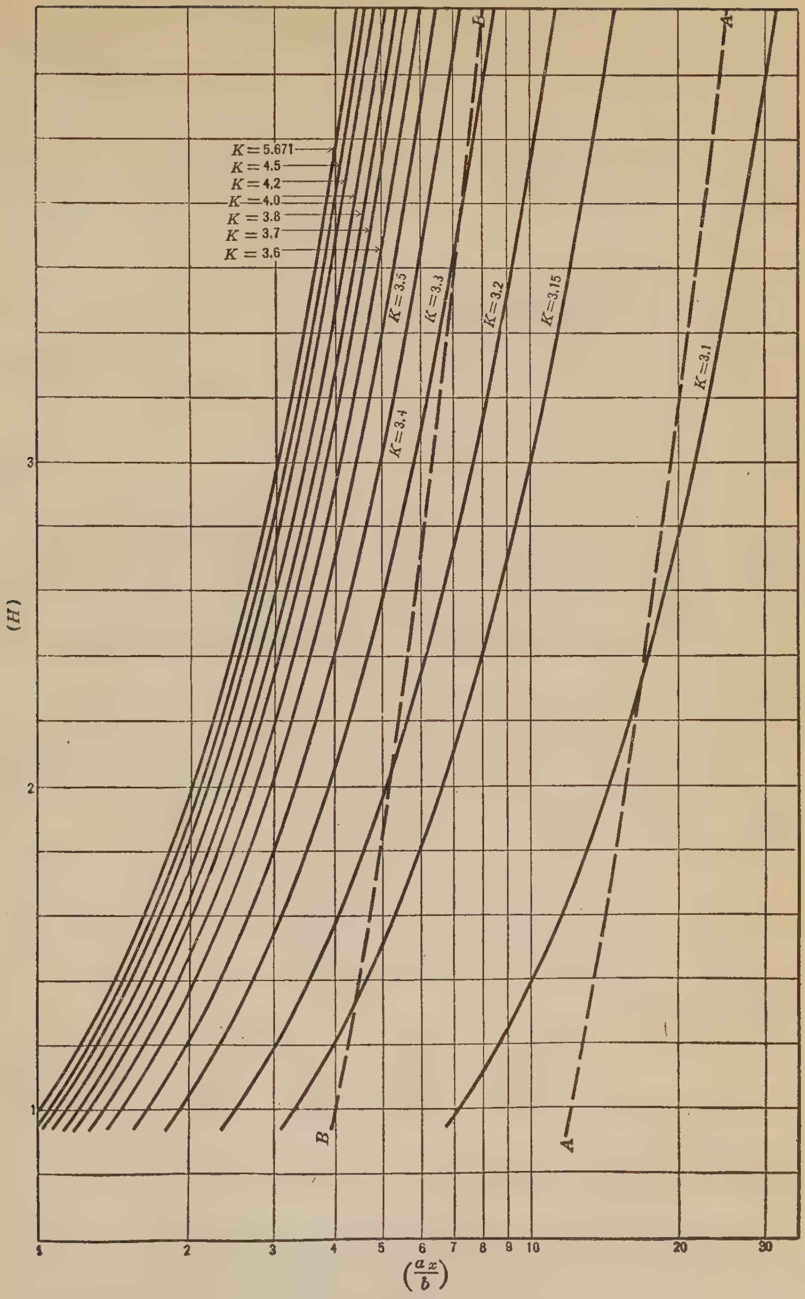
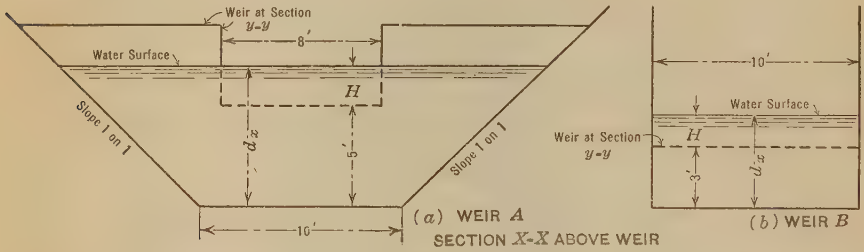


Fig. 8.—Values of K in $Q = K b H^{\frac{3}{2}}$, Assuming Uniform Velocities and no Friction.

Fig. 8 can now be applied to any weir by plotting on it a curve of coincident values of H and $\frac{a_x}{b_y}$ for that particular weir. Lines A A and B B of Fig. 8 are such curves for the weirs of Fig. 9. Fig. 8 is not put forth as a substitute for experimental determinations of the factor, K, but rather as a groundwork for such determinations and as a means of directly introducing the effect of the factor, a_x.



(a) WEIR A
SECTION X-X ABOVE WEIR

(b) WEIR B

FIG. 9.

Fig. 8 is drawn assuming that a uniform velocity exists throughout the area, a_x and likewise in a_y, and that no frictional losses occur between Sections x and y. The degree to which these assumptions are approached are, of course, governed by the details of the weir crest and of the approach channel between x and y.

TABLE 7.—COINCIDENT VALUES OF H AND $\frac{a_x}{b_y}$ IN FIG. 9.

H.	WEIR A.				WEIR B.			
	$b_y = (8 - 0.2\,H)$.	$d_x = (H + 5)$.	$a_x = d_x\,(10 + d_x)$.	$\frac{a_x}{b_y}$*	$b_y = (10 - 0.2\,H)$.	$d_x = (H + 3)$.	$a_x = 10\,d_x$.	$\frac{a_x}{b_y}$†
1	7.8	6.0	96.0	12.31	9.8	4.0	40.0	4.08
2	7.6	7.0	119.0	15.66	9.6	5.0	50.0	5.21
3	7.4	8.0	144.0	19.46	9.4	6.0	60.0	6.38
4	7.2	9.0	171.0	23.75	9.2	7.0	70.0	7.61

* Line A A of Fig. 8.
† Line B B of Fig. 8.

In general, assuming no friction, if the non-uniformity in velocity is greater at x than at y (which would be the usual condition), the K values given by Fig. 8 are too small. K is always decreased by an increase in friction, and *vice versa*. This Fig. 8 gives a definite value of K for any value of H as measured on any particular weir, on the assumption of uniform velocity distributions and no friction.

Any experimental determination of the factor, K, differing from the values given would indicate, of course, a departure from the assumptions of Fig. 8 and would give a value of Q from Equation (30), which, if substituted in Equation (29), would give a value of h_x differing from $\frac{v_x^2}{2\,g}$ $\left(v_x \text{ being the mean velocity, } \frac{Q}{a_x}\right)$.

From Equations (29) and (30),

$$h_x = k' H \text{..........................} (36)$$

in which,

$$k' = 0.4717\ K^{\frac{2}{3}} - 1 \text{......................} (37)$$

also,

$$0.4717 = \frac{3}{2\ g^{\frac{1}{3}}}$$

Coincident values of K and k' from Equation (37) are listed in Table 6, from which it is seen that $k' = 0$ only when $K = 3.087$ (its lower limit), and hence for any greater value of K there must be some velocity of approach in some part of a_x even if a_x is infinitely large. h_x from Equation (36) will equal $\frac{v_x^2}{2\,g}$ only when K in Equation (37) agrees with Fig. 8.

Line BB of Fig. 8 would indicate values of K from 3.13 to 3.34 for the weir of Fig. 9(b), as H increases from 1 to 4, assuming uniform velocity distribution and no friction, and should be correct if these assumptions were realized. Suppose experiments on this weir give a K of 3.33 for $H = 1$, instead of 3.13, indicating a correction factor applied to Fig. 8 of 1.064 $\left(= \frac{3.33}{3.13}\right)$.

Then,

$$Q = 3.33 \times 9.8 \times 1 = 32.634$$

$$\frac{v_x^2}{2g} = \frac{Q^2}{2\ g\ a_x^2} = \frac{(32.634)^2}{2\ g\ (40)^2} = 0.01035$$

and, from Equations (36) and (37),

$$h_x = 0.0518$$

Hence, in this case,

$$h_x = 5\,\frac{v_x^2}{2g}$$

An "effective" velocity head of $5\,\frac{v_x^2}{2\,g}$ would indicate a high degree of non-uniformity in the velocities at Section x, probably a high surface velocity with perhaps some dead water, or even back-flow, at the invert.

Let H_0 equal the value of H at a point where there is no velocity of approach. Then, $h_x = 0$, and, from Equation (29),

$$Q = 3.087\ b\ H_0^{\frac{3}{2}} \text{........................} (38)$$

from which,

$$H_0 = 0.4717 \left(\frac{Q}{b}\right)^{\frac{2}{3}} \qquad \text{(39)}$$

which is the elevation of the reservoir flow line above the crest of a spillway weir, assuming uniform velocity on the crest and no friction losses in approaching the weir. (Q = flow over weir and b = net width of stream on crest.)

ORIFICES

Since Equation (10) is perfectly general, it would be possible to apply its principles to an orifice. Sections x and y are regarded as being, respectively, above and below the plane of the orifice. At the plane of the orifice, there is an intermediate section, 0-0. Uniform velocity distributions are assumed at all sections.

$$Q = a_x\, v_x = a_0\, v_0 = a_y\, v_y \qquad \text{(40)}$$

Let the head loss between Sections x and y, equal,

$$F = k\, h_0 \qquad \text{(41)}$$

in which, k is a coefficient, depending on local conditions.

$$h_0 = \frac{v_0^2}{2g} = \frac{Q^2}{2\, g\, a_0^2},\quad h_x = \frac{Q^2}{2\, g\, a_x^2}, \text{ and } h_y = \frac{Q^2}{2\, g\, a_y^2} \qquad \text{(42)}$$

From Equation (10),

$$(d_x + h_x) = (d_y + h_y) + (k\, h_0 - I) \qquad \text{(43)}$$

S is the water-surface drop between x and y, and $S = d_x + I - d_y$. Hence, from Equation (43),

$$S = k\, h_0 + h_y - h_x \qquad \text{(44)}$$

From Equations (44) and (42),

$$2\, g\, S = Q^2\left(\frac{k}{a_0^2} + \frac{1}{a_y^2} - \frac{1}{a_x^2}\right)$$

and,

$$Q = a_x\, a_0\, a_y \sqrt{\frac{2\, g\, S}{k\, a_x^2\, a_y^2 + a_0^2\, a_x^2 - a_0^2\, a_y^2}} \qquad \text{(45)}$$

Compare Equation (45) with the following formula:

$$Q = c\, a_0 \sqrt{2\, g\, S} \qquad \text{(46)}$$

in which, c is a "coefficient of discharge" in terms of S.

From Equations (45) and (46),

$$k = \left(\frac{1}{c}\right)^2 - \left(\frac{a_0}{a_y}\right)^2 + \left(\frac{a_0}{a_x}\right)^2 \qquad \text{(47)}$$

$a_y = c_1\, a_0$, in which, c_1 is the coefficient of contraction, and, hence,

$$k = \frac{1}{c^2} - \frac{1}{c_1^2} + \left(\frac{a_0}{a_x}\right)^2 \qquad \text{(48)}$$

Equations (47) and (48) lead to the following special formulas:

If $a_x = a_y$,

$$k = \frac{1}{c^2} \qquad \text{(49)}$$

If the velocity of approach is zero, $a_x = \infty$, and,

$$k = \frac{1}{c^2} - \frac{1}{c_1^{\,2}} \qquad (50)$$

If c_1 in Equation (50) is 1.0,

$$k = \frac{1}{c^2} - 1 \qquad (51)$$

If c_1 in Equation (50) is 0.6,

$$k = \frac{1}{c^2} - 2.78 \qquad (52)$$

TABLE 8.—COINCIDENT VALUES OF c AND k.

Value of c.	Values of k.		
	From Equation (49).	From Equation (51).	From Equation (52).
0.5	4.0	3.0	1.22
0.6	2.78	1.78	0
0.8	1.56	0.56	....
1.0	1.0	0	....

It is to be remembered that c and k in Table 8 are coefficients, such that, $Q = c\, a_0 \sqrt{2\, g\, S}$ (Equation (46)) and $F = k\, h_0$ (Equation (41)).

Since F cannot be negative, k cannot be negative and, hence, from Equation (50), it follows that, in the case of zero velocity of approach, c cannot be greater than c_1.

From Equations (42) and (46),

$$h_0 = c^2\, S \qquad (53)$$

and, hence, from Equation (41),

$$F = k\, c^2\, S \qquad (54)$$

and,

$$k\, c^2 = \frac{F}{S} \qquad (55)$$

It may be that this discussion of orifices is more academic than useful, but it brings out some interesting relations and demonstrates the general applicability of the basic equations.

SIPHONS

In the basic equations, (6), (10), and (11), d is the vertical distance from the invert to the "hydraulic gradient". The "hydraulic gradient" (Line B, Fig. 1) is the water surface profile on the assumption of equal barometric pressures on the water surfaces at all sections.

Let D = the water head corresponding to the barometric pressure; and d', the vertical distance from the invert to the water surface. Then the basic energy equation, Equation (6), may also be expressed as:

$$d'_x + D_x + h_x + I = d'_y + D_y + h_y + F \qquad (56)$$

If D is constant, the water surface profile and the "hydraulic gradient" coincide, and d and d' are synonymous at all sections.

Subtracting Equation (6) from Equation (56),

$$d'_x + D_x - d_x = d'_y + D_y - d_y \quad \text{.................} (57)$$

Suppose the water surface coincides with the hydraulic gradient at Section x. Then, $d_x = d'_x$ and from Equation (57),

$$d'_y = d_y + (D_x - D_y) \quad \text{.....................} (58)$$

On the further supposition that $D_y = 0$

$$d'_y = d_y + D_x \quad \text{.........................} (59)$$

Equation (59) represents the conditions existing when Section x is in an open channel or pool and Section y is in a closed pipe running full. In this case, D_x is the atmospheric head and d'_y is the vertical distance from the invert to the water level in a vertical vacuum tube at Section y and is the maximum possible water depth at that section.

The theoretical value of D_x is about 34 ft., but as a matter of practice it is seldom safe to assume D_x as greater than 28 ft.

In computing a full conduit or siphon down stream from open head-water, the first application of Equation (56) would be as follows:

$$(d'_y + h_y) = (d'_x + D_x + h_x) + (I - F) \quad \text{............} (60)$$

In the next succeeding stretch of conduit, Section y becomes Section x, D_x becomes zero, and,

$$(d'_y + h_y) = (d'_x + h_x) + (I - F) \quad \text{.............} (61)$$

Since h and F are independent of d in a full conduit, Equation (61) will produce the same results as Equation (11) unless d, at some section, becomes less than the vertical depth of the conduit at that point. In that case the conduit, instead of flowing at the depth, d, will continue to flow full unless d' ($= d +$ atmospheric head) becomes less than the full conduit depth.

DISCUSSION

Frank S. Bailey,* Assoc. M. Am. Soc. C. E. (by letter).—The author seems to have accomplished his purpose, as stated in his Synopsis, "to present a workable general method for the analysis of stream flow in irregular channels". His fundamental assumptions are generally accepted as fair ones; his development of the equations showing the relations between the various factors governing the flow are logical; and, in so far as the writer has discovered, his mathematics are correct. Diagrams showing the relations of d plus h, and Q have been used for several years,† but the addition of f-curves by the author is a valuable new development.

Of the numerous papers and articles which have been published in the past with the object of determining the location of the water surface for a given rate of flow in an irregular channel, few, if any, are so comprehensive and clearly stated as that of Mr. Casler. His definition of the Chezy slope factor is commendable.

The fact that a given quantity of water can flow at different depths in a channel, may result in considerable damage if care is not exercised in the design of the channel. Suppose that a spillway channel for an earth dam is located on the side of a hill which rises from the bed of the stream, and that it is necessary to build a wall on the lower side of the channel, of sufficient height to prevent any flood flow from overtopping the wall with possible injury to the down-stream slope of the dam. A spillway channel wall might be designed on the basis that in all cases the water would flow at high velocity at a low depth and if, due to some unexpected cause, the actual velocity were lower and the depth higher than anticipated, the wall itself might act as a spillway, with undesirable consequences.

Karl R. Kennison, M. Am. Soc. C. E., mentions‡ the case of two high flood flows of not greatly different amounts which ran at different depths in a wooden flume, and in his discussion of that paper, the late Frederic P. Stearns, Past-President, Am. Soc. C. E., described§ at greater detail the same phenomenon. If a horizontal deflection or a curve exists in the channel, the chances of a mistake in the design are increased, and more knowledge on the subject of loss of head in curves and on differences in elevation of water on the inner and outer edges of curves would be helpful.

An entering or discharging stream of water at the intersection with a branch channel may also have to be taken into account. Losses due to intersecting streams have been ably treated‖ by J. C. Stevens, M. Am. Soc. C. E., from a theoretical standpoint.

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† See, "The Hydraulic Jump and Critical Depth in the Design of Hydraulic Structures," by Julian Hinds, M. Am. Soc. C. E., *Engineering News-Record,* November 25, 1920, Vol. 85, p. 1034.

‡ "The Hydraulic Jump in Open-Channel Flow at High Velocity," *Transactions,* Am. Soc. C. E., Vol. LXXX (1916), p. 338.

§ *Loc. cit.*, p. 368.

‖ "Theoretical Energy Losses in Intersecting Pipes," *Engineering News-Record,* July 22, 1926, p. 140.

The author's treatment of (*a*) flow over weirs; (*b*) the effects of height of weir above the bottom of the channel; (*c*) area of channel above the weir as affecting the velocity of approach; and (*d*) flow over the weir, is novel and ingenious.

CECIL E. PEARCE,* ASSOC. M. AM. SOC. C. E. (by letter).—This excellent paper should be welcomed by all hydraulic designers. Basically, nothing new is presented, but the treatment is exceptionally clear, and presents the facts in a readily usable form. Having set up the fundamental energy equation, which is well known and is commonly called "Bernoulli's theorem", the author bases the larger part of his mathematics upon it, of necessity. The treatment of the critical section, however, and the curves used, form valuable tools which lead to a clearer understanding and shorten the work involved in computing the water surface in an irregular channel or conduit for a given discharge.

The writer, in calculating the flood discharges through the Exchequer Dam and Power House, in 1926, used a step-by-step process, with Chezy's formula, $c = C\sqrt{r\,s}$, obtaining the proper elements by trial and error. The determination of the proper trial was based on the same fundamental reasoning as that of the author.

The conduit was very irregularly shaped. The steps were as follows:

1.—Assume a given value of Q, say, 300 sec-ft.

2.—For a first trial, assume that, with 300 sec-ft. flowing, the water will have a certain depth at the lower end. Since $Q = a\,v$, the corresponding velocity at that lower end may be computed.

3.—With this as a basis, calculate by trial and error what the water surface elevation would be at every point along the irregular conduit, and, finally, in the reservoir.

4.—Repeat this process a number of times for $Q = 300$ by starting with different assumed depths of water at the lower end.

A number of such computations gives an equal number of possibilities for the water-surface profile. Any assumed elevation of the water surface at the lower end determines the water elevation in the reservoir. Which of these profiles is the proper one?

To answer this question, consider what holds the water surface where it is. If a glass of water is emptied on a flat surface, it immediately tries to flatten out to zero thickness. The same is true of the water in a channel; all that holds the water surface where it is is the back pressure. The water, in attempting to flatten out, meets with obstacles (resistance). Considering that the only resistance causing a reaction to the flow of the water is the frictional force and the resistance of inertia in producing velocity, it is those two forces that supply the back pressure to restrain the water surface.

Also, with any given surface elevation in a reservoir, there must be one maximum value of Q which can be discharged through a given channel. Referring to Fig. 10, F represents energy consumed in overcoming friction;

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and h_y represents the energy consumed in producing velocity. Therefore, $(h_y + F)$ is the total energy consumed in overcoming friction and producing velocity.

By the theory of least work, however, the total energy consumed must be a minimum. Therefore, $(h_y + F)$ must be a minimum, and, since Points (A) and (D) are given, Point (C) must rise as high as possible. For a higher portion of Point (C) the value of Q would be less, or the elevation at Point (E) would increase.

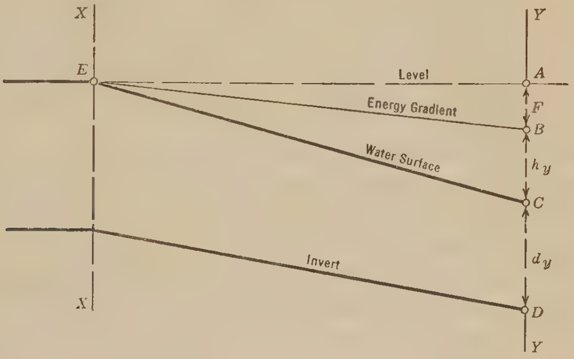

FIG. 10.

Hence, for a given maximum discharge, Q, and a given water surface elevation at Point (E), the water surface, E-C, will be as nearly level as possible, or for a given Q and the water surface as nearly level as possible (in order to comply with the minimum energy loss requirement), the water surface elevation at Point (E) will be the minimum which would discharge the given Q. This is necessary in order to make Q the maximum for that elevation.

If it is hard to visualize the theory entirely, a simple calculation will make the statement clearer:

(1) Consider a short channel leading from a reservoir.
(2) Assume a given discharge, Q.
(3) By successively taking assumed depths at the lower end, calculate what the water surface elevation in the reservoir will be in order to give that discharge.
(4) Suppose at first shallow depths are assumed at the lower end; then, in successive trials, somewhat greater and greater depths at the lower end are taken.
(5) For the first trial the very shallow depth requires a great velocity; consequently, the friction loss and velocity head loss are great, and a relatively high water elevation in the reservoir is required to furnish the necessary heads.
(6) For the next trial, with a greater depth at the lower end, the velocity is less, and consequently the friction loss is less. The elevation of the water surface in the reservoir then need not be as great as in the first trial. Finally, after repeated trials the reservoir elevation reaches a minimum, and then begins to rise

> again. Or it may be found that, with the minimum head in the reservoir, the total energy loss is a minimum in order to discharge the given Q, and that with another trial calculation, assuming the water surface at the lower end of the conduit to be a little higher than for the minimum reservoir elevation, either the water surface in the reservoir must rise to create the given Q, or the discharge, Q, will decrease. This shows, as before, that this minimum elevation in the reservoir represents the water surface profile of minimum energy loss for the specific task of discharging the given maximum Q.

In the case of the Exchequer Dam, the trial calculation giving the least elevation of the water surface in the reservoir was chosen as representing the actual condition which would obtain. For comparing these calculations with results obtained by Mr. Casler's method, the writer's computation will be presented first.

A longitudinal section through the dam, power house, and channels is shown in Fig. 11. As an aid to calculation, curves of the hydraulic radius and the area of each cross-section were plotted, but are not reproduced.

Various constants were adopted as follows:

(1) At the entrance, A, three piers project inward. The coefficient for entrance loss, K_0, was therefore taken as 0.78.*

(2) For the loss due to obstruction and sudden contraction at F, the factor, 0.25, was assumed.

(3) The coefficient, n, for friction or rough concrete was taken as 0.015.†

Further, a curve was plotted for determining the coefficient, C, for various values of hydraulic radius, r, when $S = 0.001$ and $n = 0.015$ for a rough concrete channel. The values of C for a given r vary little between the limits, $S = 0.0004$ and $S = 0.01$; so that using values of C when $S = 0.001$ (or 0.0004 or 0.01) does not materially affect the answer if the actual water slopes are within those limits.

Using these various curves, Table 9 was prepared, showing the various trial computations. Many of the notations and derivations will be apparent from Fig. 11. The terms, A, v, h, r, l, S, and H, give the area, velocity, velocity head, hydraulic radius, length of channel, hydraulic slope, and differential head, respectively. The subscripts refer to the parts of the channel concerned, as follows: 1, for Point G; 2 for the reach, G F; 3, for Point F; 4 for Point E; 5 for the reach, E-D; 6 for Point D; 7 for Point C; 8 for the reach, C-B; and 9 for Point B (all these points are numbered to correspond in Fig. 11).

The values of "Water Elevation" (Table 9, Columns (1), (6), (15), (24), (31), etc.) are assumed in each case, whereas other "computed elevations" (Columns (22), (30), etc.) are derived from the tabular values. Ordinarily, for the first trial, the elevation in Column (22) will not check that assumed in Column (15). The values given are for the final results and similarly for the other reaches.

Then, the curve in Fig. 12 was plotted from Column (73), Table 9. Evidently, the minimum elevation of the water surface is at Elevation

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* See King's Handbook, p. 152.

† *Loc. cit.*, p. 191.

436.95. This same process may be repeated for other discharges, Q, of 500, 1 000 sec-ft., etc., which will give the data for plotting another curve which would show the discharge through the channel for various elevations of the water surface in the reservoir.

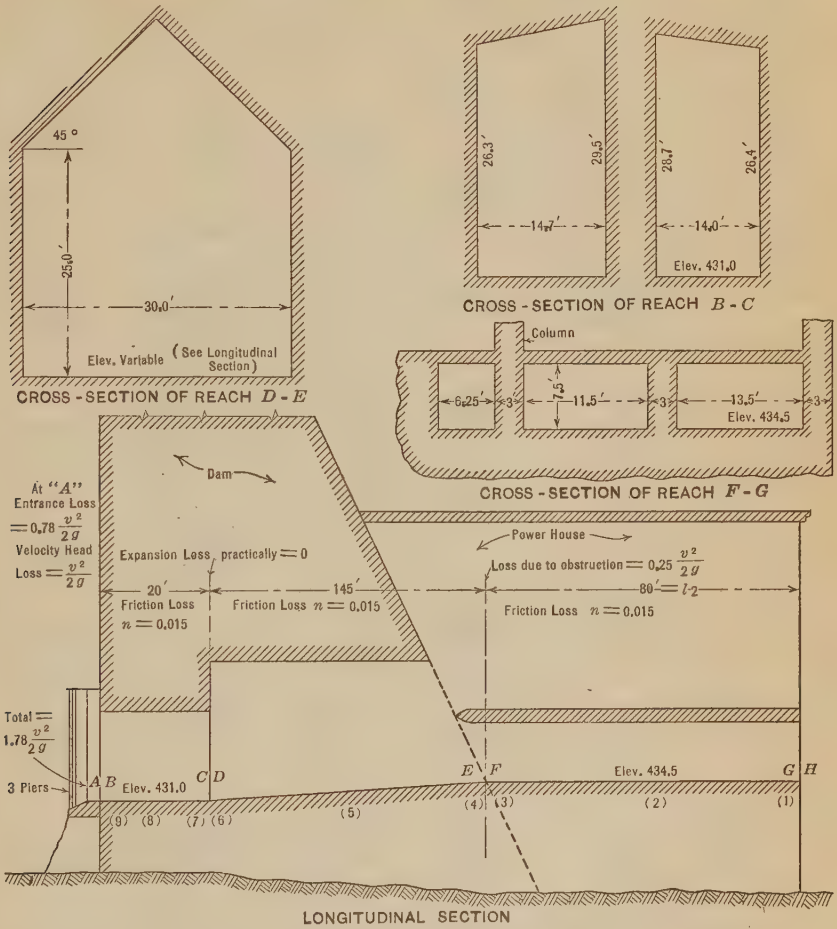
45°
25.0′
30.0′
Elev. Variable
(See Longitudinal Section)
CROSS-SECTION OF REACH D-E
26.3′
29.5′
14.7′
28.7′
26.4′
14.0′
Elev. 431.0
CROSS-SECTION OF REACH B-C
Column
6.25′
3
7.5′
11.5′
3
13.5′
3
Elev. 434.5
CROSS-SECTION OF REACH F-G
Dam
At "A" Entrance Loss $=0.78\frac{v^2}{2g}$
Velocity Head Loss $=\frac{v^2}{2g}$
Expansion Loss, practically $=0$
Power House
Loss due to obstruction $=0.25\frac{v^2}{2g}$
20′
145′
80′ $= l_2$
Friction Loss $n=0.015$
Friction Loss $n=0.015$
Friction Loss $n=0.015$
Total $=1.78\frac{v^2}{2g}$
3 Piers
A B
Elev. 431.0
C D
E F
Elev. 434.5
G H
(9) (8) (7) (6) (5) (4) (3) (2) (1)
LONGITUDINAL SECTION

FIG. 11.—SECTIONS OF FLOOD CONTROL CHANNEL, THROUGH DAM AND POWER HOUSE.

A check will now be applied to those calculations using Mr. Casler's method, to compare not only the results, but also the advantages and time saving. Fig. 13, showing values of Q and f for Section F-G, is plotted from values computed according to the author's method, for $Q = 300$ sec-ft. Similar curves were plotted for each of the other two sections, but are not presented.

By inspection, it is probable that the critical section is at (or near) the lower end. If it is, then the procedure would be as follows.

TABLE 9.—HYDRAULIC COMPUTATIONS, EXCHEQUER DAM.

(Various Subscripts Indicate the Section Concerned, as Numbered Along Line *A-H*, Fig. 11.)

SECTION *F-G.*

Water elevation, at *G*. in feet. (1)	A_1, in square feet. (2)	$v_1 = \frac{Q}{A_1}$, in feet per second. (3)	*Q*, in cubic feet per second. (4)	$h_1 = \frac{v_1^2}{2g}$, in feet. (5)	Mean water elevation, *G-F*, in feet. (6)	(r_2), in feet. (7)	A_2, in square feet. (8)	$v_2 = \frac{Q}{A_2}$, in feet per second. (9)	$\left(\frac{v_2^2}{r_2}\right)$. (10)	*C*. (11)	C^2. (12)	$S_2 = \frac{\left(\frac{v_2^2}{r_2}\right)}{C^2}$. (13)
436	46	6.52	300	0.66	436.30	1.33	56	5.35	21.6	104	10 800	0.002
437	78	3.84	300	0.23	437.10	1.73	81	3.7	7.91	110	12 100	0.000655
435.50	31	9.68	300	1.46	436.0	1.15	46	6.52	37	102	10 400	0.00356
436.50	62	4.84	300	0.36	436.57	1.47	64	4.69	14.9	107	11 400	0.0013
435.25	23	13.0	300	2.63	437.0	1.69	78	3.85	8.75	110	12 100	0.00072

TABLE 9.—(*Continued.*)

SECTION *F-G.*									SECTION *D-E.*			
l_2, in feet. (14)	Water elevation at *F*, in feet. (15)	A_3, in square feet. (16)	$v_3 = \frac{Q}{A_3}$, in cubic feet per second. (17)	$h_3 = \left(\frac{v_3^2}{2g}\right)$, in feet. (18)	$h_1 - h_3$, in feet. (19)	H_2 = friction, loss, $S_2 l_2$, in feet. (20)	Total gain of head in *G-F*, = Column (19) + Column (20), in feet. (21)	Computed elevation at *F*. Column (1) + Column (21), in feet. (22)	$H_3 = 0.25 h_3$. (23)	Water elevation at *E*. in feet. (24)	A_4, in square feet. (25)	$v_4 = \frac{Q}{A_4}$, in cubic feet per second. (26)
80	436.50	62	4.84	0.36	0.30	0.16	0.46	436.46	0.09	436.6	65	4.61
80	437.06	80	3.75	0.22	0.01	0.05	0.06	437.06	0.06	437.10	78	3.84
80	437.0	78	3.84	0.23	1.23	0.28	1.51	437.01	0.06	437.05	78	3.84
80	436.64	67	4.47	0.31	0.06	0.10	0.16	436.66	0.08	437.70	95	3.16
80	437.3	106	2.83	0.12	2.51	0.06	2.57	437.82	0.03	437.8	106	2.83

TABLE 9.—(*Continued.*)

SECTION *D-E*.

$h_4 = \left(\frac{v_4^2}{2g}\right)$, in feet. (27)	$(h_3 - h_4)$. in feet. (28)	Total gain, *F-E*, = Column (23) + Column (28), in feet. (29)	Computed water elevation at *E*, = Column (22) + Column (29), in feet. (30)	Mean water elevation, *E-D*, in feet. (31)	Mean elevation of bottom for *E-D*, in in feet. (32)	Mean depth, *E-D*, in feet. (33)	A_5, in square feet. (34)	$v_5 = \frac{Q}{A_5}$, in cubic feet per second. (35)	r_5. (36)	$\left(\frac{v_5^2}{r_5}\right)$. (37)	*C*. (38)	C^2. (39)
0.33	+0.03	+0.12	436.58	436.75	432.75	4.00	120	2.5	3.15	1.98	120	14 400
0.23	−0.01	+0.05	437.11	437.21	432.75	4.46	135	2.22	3.45	1.43	121	14 600
0.23	0.00	+0.06	437.07	437.21	432.75	4.46	135	2.22	3.45	1.43	121	14 600
0.16	+0.15	+0.23	436.89	436.95	432.75	4.20	127	2.36	3.3	1.69	120	14 400
0.12	0.00	+0.03	437.85	437.92	432.75	5.17	155	1.93	3.85	0.97	123.55	15 210

TABLE 9.—(*Continued.*)

SECTION *D-E*.										SECTION *B-C*.	
$S_5 = \frac{\left(\frac{v_5^2}{r_5^2}\right)}{C^2}$. (40)	l_5, in feet. (41)	$H_5 = S_2 l_2$, in feet. (42)	Water elevation at *D*, in feet. (43)	A_6, in square feet. (44)	$v_6 = \frac{Q}{A_6}$, in cubic feet per second. (45)	$h_6 = \left(\frac{v_6^2}{2g}\right)$, in feet. (46)	$(h_4 - h_6)$, in feet. (47)	Total gain, *E - D*, Column (42) + Column (47), in feet. (48)	Water elevation at *D* = Column (30) + Column (48), in feet. (49)	Water elevation, at *C*, in feet. (50)	A_7, in square feet. (51)
0.000137	145	0.02	436.85	176	1.70	0.04	0.29	0.31	436.89	436.9	170
0.000098	145	0.01	437.31	190	1.58	0.04	0.19	0.20	437.31	437.30	180
0.000098	145	0.01	437.31	190	1.58	0.04	0.19	0.20	437.27	437.26	180
0.000117	145	0.02	437.0	180	1.67	0.04	0.12	0.14	437.03	437.03	175
0.000064	145	0.01	438.00	210	1.43	0.03	0.09	0.10	437.95	438.95	228

TABLE 9.—(*Continued.*)

SECTION *B-C.*

$v_7 = \left(\frac{Q}{A_7}\right)$, in cubic feet per second. (52)	$h_7 = \left(\frac{v_7^2}{2g}\right)$, in feet. (53)	$h_6 - h_7$, in feet. (54)	Computed water elevation at C = Column (49)+Column (54), in feet. (55)	Mean water elevation, C-B, in feet. (56)	A_8, in square feet. (57)	$v_8 = \frac{Q}{A_8}$, in cubic feet per second. (58)	r_8, in feet. (59)	$\left(\frac{v_8^2}{r_8}\right)$. (60)	C. (61)	C^2. (62)	$S_8 = \frac{\left(\frac{v_8^2}{r_8^2}\right)}{C^2}$. (63)
1.76	0.05	—0.01	436.88	436.88	170	1.76	3.24	11.01	125	15 625	0.000065
1.66	0.04	0.00	437.31	437.31	180	1.66	3.35	0.822	121	14 600	0.000056
1.66	0.04	0.00	437.27	437.31	180	1.66	3.35	0.822	121	14 600	0.000056
1.71	0.05	—0.01	437.02	437.02	175	1.71	3.30	0.89	120	14 400	0.0000615
1.31	0.03	0.00	438.95	438.95	228	1.31	3.67	0.467	122	14 900	....

TABLE 9.—(*Continued.*)

SECTION *B-C.*

l_8, in feet. (64)	H_8, in feet. (65)	Water surface at B, in feet. (66)	A_9, in square feet. (67)	$v_9 = \frac{Q}{A_9}$, in cubic feet per second. (68)	$h_9 = \left(\frac{v_9^2}{2g}\right)$, in feet. (69)	$h_7 - h_9$ = velocity head gain from C to B, in feet. (70)	Computed water elevation at B = Column (55)+Column (70). (71)	Total loss at entrance = 1.8 h_9, in feet. (72)	Computed water elevation in reservoir = Column (71)+Column (72). (73)	Point number for plotting. (74)
20	0.00	436.88	169	1.78	0.05	0.00	436.88	0.09	436.97	1
20	0.00	437.31	180	1.66	0.04	0.00	437.31	0.07	437.38	2
20	0.00	437.31	180	1.66	0.04	0.00	437.31	0.07	437.38	3
20	0.00	437.02	175	1.71	0.05	0.00	437.02	0.09	437.11	4
20	0.00	438.95	228	1.31	0.03	0.00	438.95	0.05	439.00	5

1.—*Determination of Water Depth at Critical Section.*—

$$\frac{a^3}{b} = 0.0311\ Q^2 = 2\ 799$$

$$\frac{a^3}{b} = b^3 \times \frac{d^3}{b} = b^2 \times d^3 = 2\ 799$$

but,

$$b = 31.25, \text{ and } b^2 = 976$$

therefore,

$$d^3 = \frac{2\ 799}{976} = 2.87$$

or,

$$d = 1.42$$

$$h = \frac{d}{2} = 0.71$$

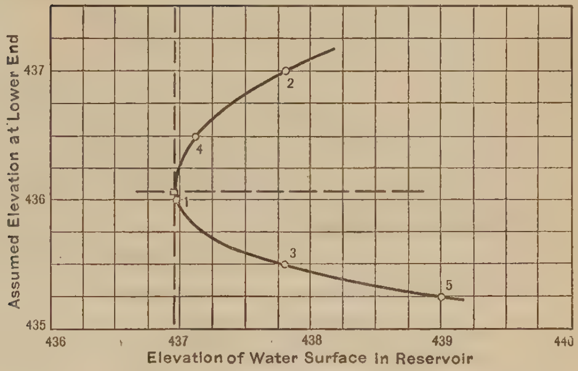

FIG. 12.—CURVE SHOWING MINIMUM ELEVATION OF WATER SURFACE IN RESERVOIR. PLOTTED FROM TABLE 9.

Therefore,

$$(d + h) = 2.13$$

Then the elevation of the water surface at G, is,

$$434.50 + 1.42 = 435.92$$

The value of f may be found from the formula, $f = \frac{p\ g}{b\ C^2}$. Since,

$$p = 31.25 + (6 \times 1.42) = 39.77$$

$$b = 31.25$$

and,

$$C = 115$$

$$f = 0.0031$$

2.—*Consideration of Reach G-F.*—The fundamental hydraulic relation between two planes, X and Y (Fig. 10), is:

$$(d_x + h_x) - \frac{l}{2} f_x = (d_y + h_y) + \frac{l}{2} f_y - l\ i$$

In this case: $l = 80$; $f_y = 0.0031$; $i = 0$; and $(d_y = h_y) = 2.13$.

Thus,

$$(d_x + h_x) - 40\,f_x = 2.13 + (40 \times 0.0031) - 0 = 2.25$$

but $f > i$; therefore, follow the Q-curve upward from the vertex (Fig. 13).

After successive trials, assume that $(d_x + h_x) = 2.31$. Then, from Fig. 13, $f_x = 0.0015$, and $40\,f_x = 0.06$; or $2.31 - 0.06 = 2.25$ (which checks, as it should).

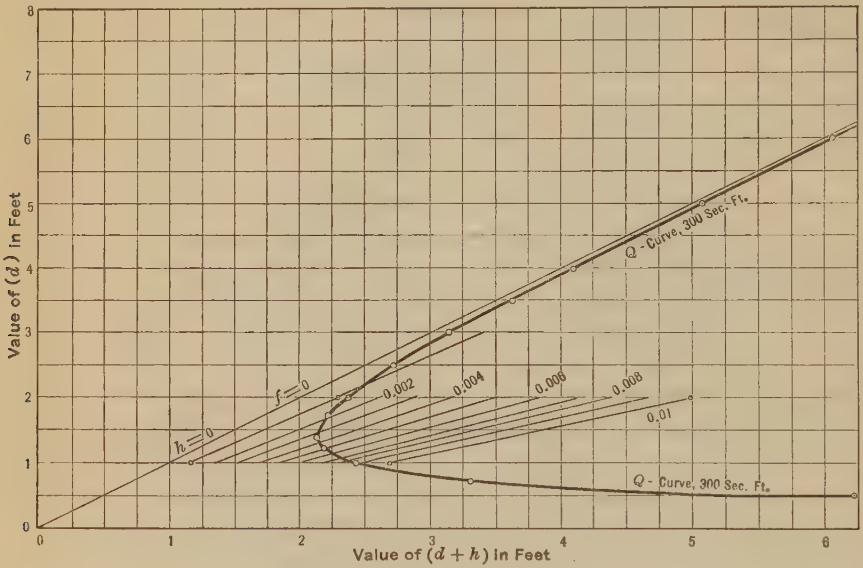

FIG. 13.—Q-CURVES FOR SECTION F-G.

Then,

$$(d_x + h_x) = 2.31$$

From Fig. 13,

$$d_x = 1.90$$

or,

$$h_x = 0.40$$

$$\text{Elevation of invert} = 434.50$$

$$d_x = 1.90$$

Therefore, water surface elevation at F = 436.40

3.—Consideration of Losses at Point E-F.—The loss due to obstruction at $F = 0.25$ times the velocity head at the point, $F = 0.25 \times 0.40 = 0.10$.

As before,

$$(d_x + h_x) - \frac{l}{2} f_x = (d_y + h_y) + \frac{l}{2} f_y - l\,i + \text{other losses}$$

$$= 2.31 + 0 + 0 + 0.10 = 2.41$$

$$i = 0,$$

and,

$$f = 0.0015$$

As $f > i$, follow the Q-curve upward from the vertex. Since $l = 0$,

$$(d_x + h_x) = 2.41$$

From a curve for Section D-E (similar to Fig. 13), $d_x = 2.03$ ft., and $f_x = 0.001$. Then,

$$\begin{array}{rl} \text{Elevation of invert at } E = & 434.50 \\ d_x = & 2.03 \\ \hline \text{Therefore, elevation of water surface at } E = & 436.53 \end{array}$$

In a similar manner, the water surface elevations (and the other concurrent functions) were calculated for each section to the reservoir.

Check on Flow at Point B.—It was found by calculation that, at B, $h = 0.05$ and $d = 5.93$. Therefore,

$$v = 1.79$$

$$a = 28.7 \times 5.93 = 170.5$$

$$Q = a\,v = 170.5 \times 1.79 = 3.05 \text{ (check)}$$

Comparison of Results.—A comparison of the water surface elevations at each point along the channel, calculated by the two different methods (Table 10), shows that one is a substantial check on the other. The writer's method gives a water surface elevation varying from about 0.08 ft. higher to 0.07 ft. lower than that of Mr. Casler.

Discussion.—Judging from experience in calculations of this kind, evidently Mr. Casler's method is much shorter and more direct, and gives a clearer vision of the whole problem. The writer's method is one of trial and error, and is very laborious.

In many hydraulic problems, the critical section may be located from a superficial glance at the physical conditions. After this section is determined, the solution, by Mr. Casler's method, is almost entirely direct.

TABLE 10.—COMPARISON OF RESULTS BY TWO METHODS.

Point.	Elevation of Water Surface.		Difference from Casler's method.
	Casler's method.	Writer's method, using Chezy formula.	
G	435.92	436.00	+0.08
F	436.40	436.46	+0.06
E	436.53	436.58	+0.05
D	436.93	436.89	—0.04
C	436.93	436.88	—0.05
B	436.93	436.88	—0.05
A	437.02	436.95	—0.07

It will be noticed that Reach F-G is a three-barreled conduit, and that these calculations assume that the water surface elevation will be the same in all three. Actually, this will not happen, for the hydraulic radii are not the same, and, therefore, the friction loss will differ. If one cared to follow out the calculations, considering each barrel separately, it could be done, but the practical difference would be negligible.

F. Theodore Mavis,* Assoc. M. Am. Soc. C. E. (by letter).—The author's use of the term, "friction factor", for the slope of the energy line is confusing since that term has long been used in referring to the measure of channel roughness, particularly in formulas for flow of fluids in pipes. The friction factor, f, in the Chezy formula, $h = f \frac{l}{d} \frac{v^2}{2g}$, is a function of the Reynolds number, $\frac{v\,d}{\nu}$, and the character of the conduit, and it is not the slope of the energy line, $\frac{h}{l}$. (h is the loss in energy head in a distance, l; d, the diameter of the conduit; v, the velocity; and ν, the coefficient of kinematic viscosity of the fluid, or its viscosity divided by its density.† Many texts refer to the slope factor in the Chezy formula and its subsequent modifications as the slope of the water surface, although this definition leads to inconsistencies which are clarified if S is defined as the slope of the total energy line.

The author states that,

> "Equations (10) and (11) are applicable to any invert profile and to any shape of channel section, constant or variable, open or closed, flowing freely or under pressure. * * * From Equations (10) and (11), the energy gradient * * * can be computed for any given channel for an assumed Q."

These equations, which are identical, are merely expressions of Bernoulli's theorem which states that the difference between the total energy $\left(\frac{p}{w} + z + \frac{v^2}{2g}\right)$, at two points on a given stream line is equal to the losses of energy between those two points. If this theorem is to be applied to any variable section one must know not only the amounts of various energy losses, but also the pressure and velocity distributions, in order to arrive at even an approximate solution of a given problem. With regard to the second part of the author's statement it may be said that nothing has been presented in the paper which will allow one to calculate the energy gradient and hydraulic gradient for flow in a rectangular channel of uniform section, in which the flow changes from torrential flow (less than critical depth) to ordinary turbulent flow (greater than critical depth), even if one neglects perimeter friction and assumes constant velocity in a given cross-section of the stream.

The writer does not agree with the statement made in the first part of "Criterion (3)", namely,

> "The general criterion for selecting the correct value of d as between these two mathematical possibilities is as follows: The water will flow at such a depth, d, that the lost head, F, between any two sections will be as nearly as possible equal to the invert drop, I, in that same stretch of channel. * * *"

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* Asst. Prof., Mechanics and Hydraulics, Univ. of Iowa, Iowa City, Iowa.

† See Walker, Lewis, and McAdams, "Principles of Chemical Engineering" (McGraw-Hill Book Co.), for values of f as a function of the Reynolds number.

Except in the very special case of steady flow at constant depth in an open channel with a uniform section, there is absolutely no general relation between the slope of the energy line and the slope of the invert of the channel. Assume, for instance, that the slope of the invert of a rectangular channel increases from 1 to 10% (in the direction of flow). The slope of the energy line below the point of intersection of these grade lines may be greater or less than the slope of the line above that point, depending on whether the water is accelerated on the steeper slope or whether it is decelerated so that the depth of flow becomes greater and greater.

The last sentence of Criterion (3), "successive values of $(d + h)$ will be as nearly as possible equal to each other", is ambiguous and would easily lead to the erroneous impression that the height of a hydraulic jump is the difference between corresponding depths of flow, d, for the same total energy $(d + h)$ as shown in the author's diagrams.

The author has stressed the energy equations of one-dimensional flow and has made no mention of the impulse and momentum equations. The latter are often indispensable in arriving at quantitative values of head losses, for instance, at sudden enlargements in section or at a hydraulic jump. These equations follow directly from the fundamental law of mechanics,

$$\frac{d}{d\,t}\,(m\,v) = F \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (62)$$

Since the mass of water moving between two cross-sections of a channel is,

$$m = \frac{Q\,w}{g}\,d\,t$$

and, assuming steady flow, Q is constant,

$$\frac{d\,(m\,v)}{d\,t} = \frac{Q\,w}{g}\,(v_1 - v_2) = F \ldots\ldots\ldots\ldots\ldots\ldots (63)$$

in which,

Q = rate of discharge;
w = weight of a unit volume of water;
g = acceleration of gravity;
v_1 and v_2 = velocities at Sections 1 and 2, respectively; and
F = the resultant of forces acting on Sections 1 and 2.

Finally, if b is the width of the water prism at a depth, z, below the free surface of the water and d_1 and d_2 are the depths of flow at Sections 1 and 2,

$$F = w \int_0^{d_2} z\,b\,d\,z - w \int_0^{d_1} z\,b\,d\,z = \frac{Q\,w}{g}\,(v_1 - v_2) \ldots\ldots (64)$$

For a rectangular channel of constant width, b, $\left(v_1 = \frac{Q}{b\,d_1}, v_2 = \frac{Q}{b\,d_2}\right)$, Equation (64) (dividing through by w) becomes,

$$d_2^2 - d_1^2 = \frac{2\,Q^2}{b\,g}\left(\frac{1}{d_1} - \frac{1}{d_2}\right)$$

or,

$$d_1\,d_2\,(d_1 + d_2) = \frac{2\,Q^2}{b\,g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (65)$$

Equation (65) expresses the relation between the depths, d_1 and d_2, above and below a hydraulic jump[*] in a rectangular channel assuming "one-dimensional" flow and neglecting friction along the walls of the channel. The energy loss due to the jump is, therefore, $\left(d_1 + \frac{v_1^2}{2\ g}\right) - \left(d_2 + \frac{v_2^2}{2\ g}\right)$.

One would infer from the treatment of the critical section and from Criterion (4) that a critical stage must be found in a given channel before the calculations of flow referred to by the author can be made. In any given problem which is capable of solution, if the depth of flow, the discharge, and the properties of the channel are known at any point the computations referred to can be carried through for conditions of one-dimensional flow. It happens that, at a critical section, the depth of flow is determinable, but the existence of a section where the velocity passes through the "critical" is rather the exceptional than the usual case. If the velocities exceed the critical, that is, if a critical section exists, the computer will have need for other tools in addition to Equations (10) and (11).

The only difficulties in problems of one-dimensional flow are to find the magnitude of the losses. With the possible exception of shock losses at sudden enlargements of section, which the writer has mentioned, the solution of these difficulties can be found only by experiment. The results of many experiments (for example, experiments on flow of water in pipes and open channels, flow through orifices and over weirs), have been generalized so that they can be used readily. Only elementary mathematical tools are necessary in solving problems of one-dimensional flow.

By no means, however, may all the problems of flow in open channels be considered to be one-dimensional flow problems even for purposes of engineering design. If the problem is one of two or three-dimensional flow the engineer will probably avoid the methods of mathematical hydrodynamics in its solution and rely almost wholly on experimental methods. Model experiments in the hydraulic laboratory should be emphasized as one of the most useful methods and one generally applicable for studying those problems of hydrodynamics and stream flow which thus far have defied mathematical analysis.

MORROUGH P. O'BRIEN,[†] ASSOC. M. AM. SOC. C. E. (by letter).—Although at first glance, Mr. Casler's method of computing non-uniform flow seems to be quite simple, it is, with its several criteria, more complicated to apply than the methods of many other writers who have treated the subject, notably Flamant,[‡] A. H. Gibson,[§] and more recently Tylvad,[‖] Schonweller,[¶] Böss,[**]

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[*] For data on hydraulic jump phenomena, see, Julian Hinds, M. Am. Soc. C. E., *Engineering News-Record*, 1920, v. 85, p. 1034; Böss, "Berechnung der Wasserspiegellage" (*Forschungsarbeiten*, Heft 284, V. D. I. Verlag, Berlin); Koch-Carstanjen, "Bewegung des Wassers" (Springer, Berlin); Reports of the Miami Conservancy District; Safranez, "Wechselsprung und die Energievernichtung des Wassers," *Der Bauingenieur*, December 3, 1927.

[†] Asst. Prof., Mech. Eng., Univ. of California, Berkeley, Calif.

[‡] Flamant, "Hydraulique."

[§] A. H. Gibson, "Hydraulics and Its Applications."

[‖] K. Tylvad, "Graensvaerdier for Vandets Hastighed ved Strommende Vandbevaegelse," *Ingenioren*, January 28, 1928.

[¶] G. Schonweller, "Beregning af Vandforing med frit Vandspeil," *Ingenioren*, January 28, 1928.

[**] Böss, "Berechnung der Wasserspiegellage," *Forschungsarbeiten*, Heft 284, V. D. I., Berlin, 1927.

Koch,* and Lindquist.† Of the formulas proposed probably the most general is that of Lindquist for channels of constant cross-section:

$$d\,(d) = \frac{1}{2}\,\frac{\zeta_n \left(\frac{d}{d_n}\right)^3 - \zeta}{\left(\frac{d}{d_c}\right)^3 - 1}\,d\,l \ldots\ldots\ldots\ldots\ldots\ldots (66)$$

This includes all the necessary criteria since d_n is the depth for normal or uniform flow, d_c is the critical depth, and ζ_n and ζ are factors depending on the slope of the energy gradient at uniform flow, and at the flow considered, respectively. The values of ζ to be used are to be obtained from the formulas:

$$\zeta = \frac{2\,g}{\sqrt[3]{R}\;M^2}$$

for the higher alternate stage, and,

$$\zeta = a + \frac{b}{\sqrt[3]{\frac{R\,v}{\nu}}}$$

for the lower alternate stage.

The constants, M, a, and b, are to be found in special tables† and ν is the kinematic viscosity. The differentiation between the frictional losses at the higher and lower stages of flow resulted from a study of some of Bazin's experiments, which showed that the manner of flow was essentially different at the two stages and that at the lower stage the loss is practically the same as for the towing of flat plates held parallel to the direction of motion.‡

An even more general formula is that of Freytag which includes terms representing the change in the bottom width and the side slope,

$$\frac{d\,z}{d\,l} = \frac{\frac{v^2}{C^2\,R} \mp \frac{Q^2}{g\,a^3}\cdot\frac{\delta\,a}{\delta\,c} \times i \mp \frac{Q^2}{g\,a^3}\left(\frac{\delta\,a}{\delta\,\alpha} \times \frac{d\,\alpha}{d\,l} + \frac{\delta\,a}{\delta\,b}\frac{d\,b}{d\,l}\right)}{1 \mp \frac{Q^2}{g\,a^3}\cdot\frac{\delta\,a}{\delta\,d}} \ldots . (67)$$

in which,

α = the side slope;
b = the width of the bottom; and
z = the elevation of the water surface.

The negative sign is to be used for accelerated, and the positive sign for decelerated, flow.

Although Equations (66) and (67) appear to be far more complicated than those given by Mr. Casler, they are equally simple to apply since in a practical case the differentials could be replaced by finite increments. They have the advantage that in some cases all the variable quantities can be expressed in terms of the distance* and the exact equation for the water surface can be obtained by integration.†‡

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* Koch and Carstanjen, "Bewegung des Wassers," Berlin, 1926.

† E. Lindquist, "Anordningar för effektiv energiomvandling vid foten av överfallsdammar," Anniversary Vol., Royal Technical Univ., Stockholm, 1927.

‡ E. Lindquist, "Om Modellregler eller Likformighetssatser vid Vattenbyggnadstekniska Försök," *Teknisk Tidskrift*, 1925.

The author indicates a very common error in the use of the Chezy-Eytelwein equation, namely, that the slope to be used is either the slope of the bottom or of the water surface. However, even when this slope factor is correctly taken to be the slope of the energy gradient, the question arises as to whether the same coefficients may be used in computing it for non-uniform flow. Böss§ made a few experiments on this problem and found that for both accelerated and decelerated flow, the coefficient, n, in the Kutter formula was the same, but his experiments were few in number and were made in a very small channel. It would seem that for an equal mean velocity in the same channel, the drop in the energy line would be greater during decelerated than during accelerated flow. In his method of computation, Koch includes an impact loss for decelerated flow.

In most computations of flow, the velocity head is based on the mean velocity without making a correction for the effect of the unequal distribution of velocity. The exact expression for h is,

$$h = \alpha \frac{\bar{v}_m^{\,2}}{2\,g} = \frac{\int_0^A \rho\, v^3\, d\, a}{2\,g \int_0^A \rho\, v\, d\, a} = \frac{\int_0^A \rho\, v^3\, d\, a}{2\,g\,.\,\rho\, \bar{v}_m\, A}\,;\ \alpha = \frac{\int_0^A v^3\, d\, a}{v_m^{\,3}\, A} \ldots\ldots (68)$$

in which,

ρ = the density;
v = the velocity over a small element of area, $d\,a$;
A = the total area of the cross-section; and
$\bar{v}_m$ = the mean velocity.

TABLE 11.—COMPARISON OF BAZIN EXPERIMENTS WITH THOSE OF NIKURADSE.

	$Q\left(\frac{m^3}{\text{sec}}\right)$.	$d\ (m)$.	$v\left(\frac{m}{s}\right)$.	$d_c\ m$.	$\frac{d}{(d+h)}$.	a.
Bazin*, Series 10..	1.236	0.265	2.318	0.356	0.497	1.161
Nikuradse†.......	7.59×10^{-3}.	0.245	0.206	0.143	0.992	1.138

* Bazin, "Recherches Hydrauliques."

† Nikuradse, "Untersuchung über die Geschwindigkeitsverteilung in turbulenten Strömungen," *Forschungsarbeiten*, Heft 281, V. D. I., Berlin.

There is some dispute as to whether the value of α is the same for both the higher and lower alternate stages of flow. The values given in Table 11, found by the graphical method of Rehbock,‖ indicate that it is the same

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* Flamant, "Hydraulique."

† K. Tylvad, "Graensvaerdier for Vandets Hastighed ved Strommende Vandbevaegelse," *Ingenioren*, January 28, 1928.

‡ G. Schonweller, "Beregning af Vandforing med frit Vandspejl," *Ingenioren*, January 28, 1928.

§ Böss, "Berechnung der Wasserspiegellage," *Forschungsarbeiten*, Heft 284, V. D. I., Berlin, 1927.

‖ Th. Rehbock, "Die Bestimmung der Lage der Energielinie bei fliessenden Gewassern mit Hilfe des Geschwindigkeitshöhen-Ausgleichwertes," *Der Bauingenieur*, 1922.

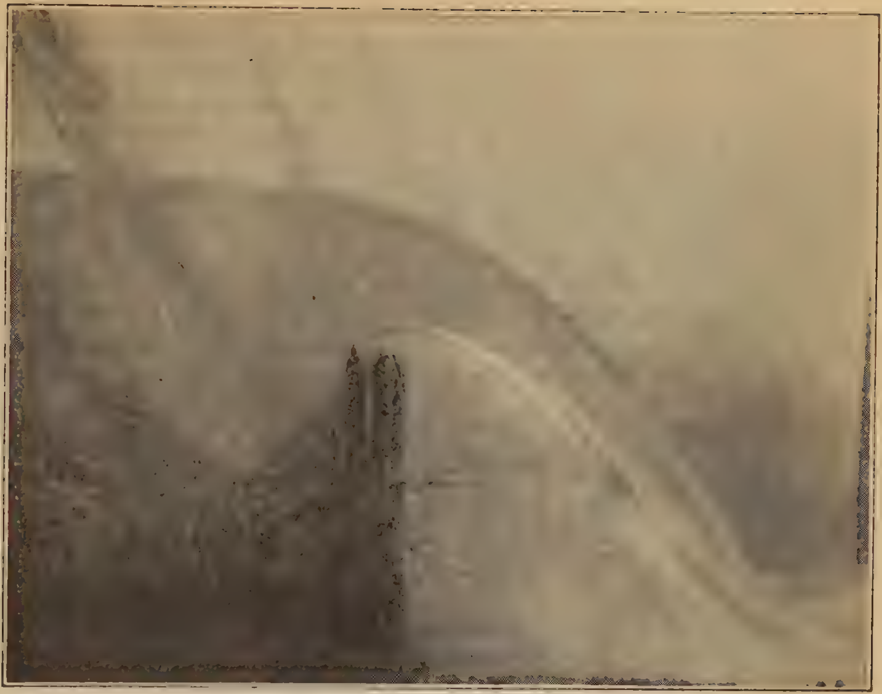

FIG. 14.—VIEW SHOWING CRITICAL DEPTH.

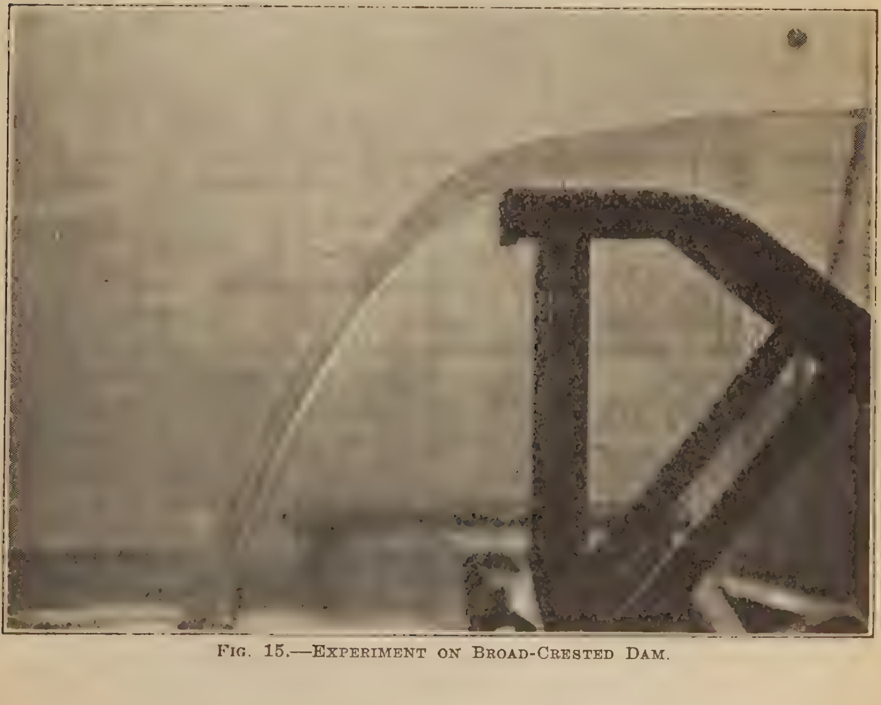

FIG. 15.—EXPERIMENT ON BROAD-CRESTED DAM.

in channels in which the distribution of velocity is determined by the friction at the walls and not by obstructions.

If it is necessary that α be included, an average value of 1.15 may be used. For the Bazin experiment (Table 11), an error of about 6% would have occurred in $(d + h)$, if α had been neglected, but for the Nikuradse experiment, the error would have been inappreciable. If the flow is disturbed by sills or other obstructions, α may have a value of 2 or even more.

Although the expression for the critical depth may be obtained directly from such general formulas as those of Gibson and Freytag* from the condition of a vertical water surface, the usual method is to base it on the condition of a maximum discharge for a certain energy content, as was done by the author. The total energy passing any section in unit time is,

$$E_s = \frac{\alpha \rho Q^3}{2 a^2} + \rho g d Q \ldots\ldots\ldots\ldots\ldots\ldots \quad (69)$$

in which, the datum for the potential energy is the lowest point in the section considered. Expressing the width of the channel as $b = F(d)$, the general condition for the critical depth is,

$$F(d) = \frac{\left[\int_0^d F(d)\, d(d)\right]^3 g}{\alpha Q^2}$$

or,

$$b = \frac{a^3 g}{\alpha Q^2}$$

For a channel of rectangular section this gives,

$$d_c = \sqrt[3]{\frac{\alpha Q^2}{g b^2}}$$

and for a parabolic section,

$$d_c = \sqrt[4]{\frac{\alpha Q^2}{\frac{8}{27} p^2 g}}$$

in which, p is the constant of the parabola.

However, this derivation does not seem to be rigorously correct since it assumes that the potential and kinetic energy are mutually convertible and that the water surface remains horizontal in passing a controlling section. Considering any transverse section of a stream, the height of the energy line is a maximum somewhere near the center; is at nearly the same height over most of the width; and drops abruptly to the water surface at the banks. The available energy of any vertical water column is the vertical distance from the energy line to the bottom and not the vertical distance to the lowest point in the cross-section. Consequently, it would seem that, under some conditions, the critical depth might not occur at all points of a cross-section.

A very simple method of representing the relation between the height of the energy line and the quantity of flow has been developed by Koch.†

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* L. Freytag, "Der Wasserabfluss in Flossgassen u. ähnlichen Gerinnen," *Forschungsarbeiten*, Heft 235, V. D. I., Berlin.

† Koch and Carstanjen, "Bewegung des Wassers," Berlin, 1926.

From the equations for the critical depth and the flow at any depth, the relation,

$$\frac{Q}{Q_{\text{max.}}} = \sqrt{\frac{27\,\alpha_{\text{max.}}\,(1 - n)\,n^2}{4\,\alpha}} \qquad \text{(70)}$$

may be obtained in which it is assumed that $(h + d)$ is a constant of known value. The quantity, n, is the ratio, $\frac{d}{(h + d)}$; $\alpha_{\text{max.}}$ and α are the correction factors used in computing h at the critical depth and the depth considered, respectively. The following tabulation gives the values of n and the ratio, $\frac{Q}{Q_{\text{max.}}}$:

n.	$\frac{Q}{Q_{\text{max.}}}$	n.	$\frac{Q}{Q_{\text{max.}}}$
0.00	0.00	0.67	1.00
0.1	0.247	0.8	0.930
0.2	0.465	0.9	0.740
0.3	0.660	0.95	0.551
0.4	0.806	1.00	0.00
0.5	0.919		

Plotting these values along a vertical line between $(d + h)$ and $d = 0$ gives a diagram which is very convenient in studying the flow through constricted sections, especially where the change in the area of the stream is very abrupt, as at bridge piers and broad, submerged weirs.

In the application of his method to the computation of the coefficients of discharge of weirs and orifices, the author makes several assumptions that are not correct. He assumes, for instance, that the velocity is uniform throughout the section above the crest of a sharp-crested weir; that the critical depth occurs above the crest of a weir; and that at a section, y, beyond an orifice, the thickness of the jet may be used as d_y in Equation (42).

Table 12 gives values of the head, depth over the crest, and critical depth for the weir shown in Fig. 14, which was tested in the Hydraulic Structures Laboratory of the Royal Technical University at Stockholm, Sweden.

TABLE 12.—TEST ON WEIR AT ROYAL TECHNICAL UNIVERSITY, STOCKHOLM, SWEDEN.

H, in meters.	$Q\left(\frac{m^3}{\text{sec}}\right)$.	d_c, in meters.	d (crest), in meters.	μ.
0.05	0.021	0.035	0.04	0.636
0.15	0.110	0.135	0.122	0.636
0.25	0.242	0.180	0.22	0.649
0.40	0.425	0.2685	0.34	0.677

The critical depth is seen to be considerably less than the depth over the crest. In Figs. 14 and 15, the horizontal lines are 5 cm., and the vertical lines are 10 cm., apart. In Fig. 14, $H = 0.25$ m., $Q = 0.242$ cu. m. per sec.

per meter of crest, and $d_c = 0.180$ m. Fig. 15, in which $Q = 0.080$ cu. m. per sec. per meter of crest, shows one of a series of experiments on the discharge coefficients of broad-crested dams. The critical depth, which was about 0.0863 m., occurs 14 cm. from the down-stream end of the crest. An examination of other photographs from this series of experiments showed that for this particular dam the critical depth occurs at approximately the same point for all discharges within the range of the experiments.

If the critical depth is uniquely defined by the discharge, then it is permissible to state that the depth over the crest of a weir is a critical depth. However, this critical depth is not the same as that used in open channels where the paths of the fluid particles are sensibly parallel.

There does not seem to be any lack of methods for computing non-uniform flow in conduits, but rather a lack of sufficient coefficients for computing losses due to impact and friction and data regarding the formation of jumps, bores, and other phenomena. Since Bazin's experiments there have been some additions to the available information made by American and European laboratories, but it is insufficient and even what has been done, is not widely known. This problem should be carefully studied in a laboratory by means of small-scale experiments supplemented by measurements of large-sized streams and canals. The establishment of a National hydraulic laboratory would provide the means for the study of this and hundreds of other similar problems and would help to remove much of the uncertainty which now arises in nearly all hydraulic computations.

R. D. N. SIMHAM,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper furnishes a broad and concise outline of many complex but closely inter-related factors to be considered in determining flow in channels, etc. The whole question is of great importance; and the fact that engineers are accustomed to apply the results of small laboratory experiments to the calculation of large works, without actually knowing the general law, and without, in fact, making proper allowances for the results obtained, makes it imperative that the question should be thoroughly investigated in the light of modern hydrostatic and hydrodynamic theories. Probably no radical advance in the theory or practice of experimental hydraulics will be made until the general question concerning the motion of water in channels, conduits, etc., is correctly solved.

For some time, the writer has been studying this problem seriously and in many ways he finds that his results and conclusions coincide fairly well with those of the author.

Derivation of the Chezy Formula.—The author assumes, as usual, that the unit of frictional resistance is proportional approximately to the square of the velocity. His assumption that the total frictional resistance of a quantity of water flowing for 1 sec. and moving forward 1 ft. is $K\,p\,v^3$ and that the energy consumed in the same is also $K\,p\,v^3$, although it would appear to be mathematically infallible, still conveys no practical idea. The value, $K\,p\,v^3$, which is the loss of energy due to frictional resistance against the kinetic motion of water, should, in fact, be equal to $\frac{W}{2\,g}\,(v_x^2 - v^2)$ when $S = 0$ and v_x represents velocity at a section 1 ft. back; then:

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$$K\,p\,v^3 = \frac{W}{2\,g}\,(v_x^2 - v^2) \qquad (71)$$

Also,

$$K\,p\,v^3 = \frac{w\,a\,v}{2\,g}\,(v_x^2 - v^2) \qquad (72)$$

By simplifying Equation (72) and transposing terms,

$$v_x^2 = v^2\left(\frac{2\,K\,p\,g}{w\,a} + 1\right) \qquad (73)$$

and when $v_x = \sqrt{2}\,v$, $2\,K\,p\,g = w\,a$, and thus,

$$K = \frac{w\,a}{2\,g\,p} \qquad (74)$$

Substituting Equation (74) for the value of K in Equation (1) and solving for v, the result is:

$$v = \sqrt{2\,f\,g} \qquad (75)$$

that is,

$$C = \sqrt{\frac{2\,g}{r}} \qquad (76)$$

Obviously, Equation (76) does not take into account any constant for friction; hence, in a case like the foregoing the author's result appears to be incomplete.

Nearly all formulas for velocity are identical in that the hydraulic radius and the sine of the angle of inclination of the stream flow are both functions of the velocity, and are generally reducible to the elementary form, $v = C\sqrt{r\,f}$. Although it is far from complete, the following list gives some of the values of C, as used by various authorities in particular cases: Chezy, 100; Downing, 100; Leslie, 100 and 68; Taylor, 100; Pole, 100; Blackwell, 95.8; Hawksley, 96.1; Bartlet, 95.9; Young, 84.3; Kirkwood, 80.0; d'Aubisson, 100 and 68; Beardmore, 100 and 94.2; Neville, 93.3 and 92.3; Stevenson, 96.0 and 69.0; and Eytelwein, 100.0 and 93.4.

Even all the more complex expressions for velocity can be reduced in some way to the terms of the simple formula, $v = C\sqrt{r\,f}$, in which it will appear that the factors, v, $\sqrt{r}$, and $\sqrt{f}$, all bear a consistent relation one to another, and that the coefficient, C, is really the involved factor through which the value of v is independently modified. To make this clear, note that the complex terms within the brackets in D'Arcy's formulas, Kutter's formula, and Bazin's formulas are the only features in which any of these expressions differ; in short, the involved term in each case only expresses the value of C as a coefficient of flow. The factors, r and f, are usually determined by observation, while on the other hand the coefficient, C, is empirical in character and its values, as used in the respective formulas, are presumed also to have been determined by experiment. In all the equations for velocity, C is the vulnerable factor and through it are introduced all the serious discrepancies that appear.

When a body, whether solid, liquid, or gaseous, is caused to slide over a surface of another, or pass through a solid enclosure, a resistance to sliding or motion is experienced which is known as the friction between the two bodies. Proceeding on this definition, the following laws of friction may be stated:

1.—Frictional resistance is proportional to the coefficient of friction applied to the surface of contact or the solid enclosure.

2.—Fractional resistance varies directly as the resultant head contributing to the movement of water, which is sometimes represented as equal to the square of the velocity of flow; and

3.—Frictional resistance is proportional to the area of the wetted surface past which the water flows.

These assumptions lead to the general relation expression:

$$l\,p\,c\,w\,(H - F) = w\,F\,a \qquad (77)$$

in which, $(H - F)$ is the total resultant kinetic head; l is the length of the channel; and c is the coefficient of friction. Therefore,

$$F = \frac{c\,l}{r}(H - F) = \frac{H}{\left(\frac{r}{c\,l} + 1\right)} \qquad (78)$$

Furthermore,

$$v = \frac{\sqrt{2\,g}}{\sqrt{c\,l}}\sqrt{r\,F} \qquad (79)$$

Assuming that the friction head is lost at a uniform rate,

$$v = \frac{\sqrt{2\,g}}{\sqrt{c}}\sqrt{r\,f} = C\sqrt{r\,f} \qquad (80)$$

in which, $C = \dfrac{\sqrt{2\,g}}{\sqrt{c}}$.

When the assumption is made that the gradient factor in the Chezy formula represents either the hydraulic gradient or the invert gradient, the question of applying the formula is abortive, or assessing the value of C is rendered practically difficult. Perhaps the proper value of the coefficient in any particular case cannot be determined by calculation or by any arbitrary rule. Presumably, even the modern complicated expressions used to determine the value of C will not faithfully take into account all the factors that influence the flow of water. In hydraulic problems, no doubt, one can hardly hope to express a complex proposition clearly with a simple formula, and, for that reason, all critical and trustworthy equations relating to the flow of water are, of necessity, somewhat involved.

If the gross energy head, $H + d$, equals M, and the net kinetic head, $H - F$, equals D, then substituting these values in Equation (78):

$$F = \frac{c\,l}{r}\,D = \frac{M - d}{\dfrac{r}{c\,l} + 1} \qquad (81)$$

The gross kinetic head is,

$$H = D\left(1 + \frac{c\,l}{r}\right) \qquad (82)$$

The net kinetic head is,

$$D = \frac{H}{\dfrac{c\,l}{r} + 1} \qquad (83)$$

The maximum resultant velocity is:

$$\sqrt{2\,g\,D} = \sqrt{\frac{2\,g\,H}{\left(\dfrac{c\,l}{r} + 1\right)}} \qquad (84)$$

The maximum discharge is:

$$Q = a\sqrt{\frac{2\,g\,H}{\left(\dfrac{c\,l}{r} + 1\right)}} \qquad (85)$$

Based on Equations (81) to (85), other equations for the critical values applied to rectangular sections (which may be compared with those of the author) may be written as follows:

(*a*) When a, the cross-section of the stream, is constant, the equations at the critical-economic section ($b = 2\,d$) take the form, $v = \sqrt{\dfrac{2^{\frac{3}{2}}\,g\,H\,a^{\frac{1}{2}}}{(\sqrt{2\,a} + 3\,l\,c)}}$, and $Q = \sqrt{\dfrac{2^{\frac{3}{2}}\,g\,a^{\frac{5}{2}}\,H}{(\sqrt{2\,a} + 3\,c\,l)}}$. Under these conditions, when $l = 0, v = 6.755\,H^{\frac{1}{2}}$ and $Q = 6.755\,a\,H^{\frac{1}{2}}$, respectively.

At the critical-efficient section $\left(D = \dfrac{d}{4}\text{ and } b = 2\,d\right)$, $v = \sqrt{\dfrac{2}{2^{0.25}\,a^{0.25}}}$; $Q = \dfrac{g^{0.5}\,a^{1.25}}{2^{0.75}}$; and $F = \dfrac{3\,c\,l}{8}$.

(*b*) Generally, when $b = n\,d$, $D = \dfrac{d}{4}$; $v = 4.013\left(\dfrac{a}{n}\right)^{0.25} = 4.013\,d^{0.5}$ and,

$$Q = a\sqrt{\frac{g\,d}{2}} = 4.013\,a\,d^{0.5} = 4.013\,\frac{a^{1.25}}{n^{0.25}} \qquad (86)$$

These will apply to channels of triangular section and trapezoidal section with some modification.

(*c*) When b is constant, $D = \dfrac{d}{2}$, and,

$$v = \sqrt{g\,d} = 5.674\,\sqrt{d} \qquad (87)$$

Then,

$$Q = g^{0.5}\, b\, d^{1.5} = 5.674\, b\, d^{1.5}$$

$$d = \frac{Q^{0.67}}{2^{0.33}\, b^{0.67}}$$

and,

$$F = \frac{c\, l\, (b^{1.67}\, g^{0.33} + 2\, Q^{0.67})}{2\, g^{0.33}\, b^{1.67}} \quad \text{.......................} (88)$$

With reference to Equation (87), there are data available to show how certain canals in America have not scoured appreciably even under relatively high velocities. The magnitude of these velocities is known from measurements of the canals, but the exact value of v that would start the scour has never been determined. Presumably, a higher velocity is required to start the banks to scour than to continue it when once started, as illustrated by the fact that certain canals do not scour even if the mean velocity is several times that suggested by Dubuat or Kennedy. The value given by Equation (87) seems to determine the greatest velocity (which is also the economical velocity) that a particular canal would stand before beginning to scour (see Table 13).

TABLE 13.—SCOUR OBSERVATIONS TO DETERMINE SCOURING EFFECTS IN THE GREAT MIAMI VALLEY.

Location.	Recorded mean depth, in feet.	Recorded maximum average velocity, in second-feet.	Computed velocity from Equation (87) assuming rectangular section.	Results.
"Big Four" Railroad Bridge, Buck Creek..................................	6.7	13.7	14.7	No scour
"Big Four" Railroad concrete arch, Tawara Creek, east of Sidney.......	8.2	23.1	16.2	Scour
Seven-Mile Creek, Ohio................	2.58	5.44	9.1	No scour
Miami River cut-off channel..........	1.8	5.5	7.6	No scour

Equations for the critical values applied to conduits with circular cross-sections flowing full may be written as follows: $D = \frac{d}{4}$; $Q = 3.149\, d^{2.5}$; and the economical diameter for any discharge, equals $d = \frac{Q^{0.4}}{3.149^{0.4}} = 0.633\, Q^{0.4}$.

The economical velocity equals,

$$v = 2\, g\, D^{0.5} = 4.01\, d^{0.5} \quad \text{.......................} (89)$$

Formulas for critical net energy head are: $d + D = 0.791\, Q^{0.4}$; $H = 0.158\, Q^{0.4} + \frac{c\, l}{4}$; $M = 0.791\, Q^{0.4} + \frac{c\, l}{4}$; and $F = c\, l$.

Table 14 shows how the economical velocities, computed from Equation (89), compare with the values given by Fanning and Unwin.

TABLE 14.—COMPARISON OF VALUES FOR ECONOMICAL VELOCITY.*

Diameter, in inches.	4	6	12	18	24	36
Equation (89)	2.3	2.8	4.0	4.9	5.6	6.9
Fanning	2.5	2.8	3.5	4.5	5.3	7.0
†Unwin ($V = 1.45\, d + 2$)	2.5	2.7	3.5	4.2	4.9	6.4

* The velocity is given in feet per second.

† d = diameter, in feet.

General equations for orifices may be written as follows:

For rectangular orifices (free, as shown in Fig. 16(*a*)):

$$Q = \frac{8.02\, b_1\, d_1\, (M - d_1)^{0.5}}{\left\{ 1 + \dfrac{2\, c\, (b_1 + d_1)}{b_1\, d_1} \right\}^{0.5}} \quad \text{(90)}$$

For rectangular orifices (submerged, as shown in Fig. 16(*b*)):

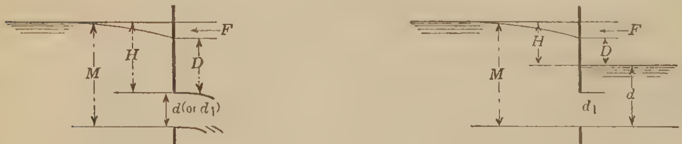
$$Q = \frac{8.02\, b_1\, d_1\, (M - d)^{0.5}}{\left\{ 1 + \dfrac{2\, c\, (b_1 + d_1)}{b_1\, d_1} \right\}^{0.5}} \quad \text{(91)}$$

For circular orifices (free, as shown in Fig. 16(*a*)):

$$Q = \frac{6.301}{(d_1 + 4\, c)^{0.5}}\, d_1^{2.5}\, (M - d_1)^{0.5} \quad \text{(92)}$$

For circular orifices (submerged, as shown in Fig. 16 (*b*)):

$$Q = \frac{6.301}{(d_1 + 4\, c)^{0.5}}\, d_1^{2.5}\, (M - d)^{0.5} \quad \text{(93)}$$



(*a*) FREE ORIFICE

(*b*) SUBMERGED ORIFICE

FIG. 16.

The formulas do not take cognizance of the velocity of the water approaching the orifice. Where this is appreciable it must be taken into account, and H or M would then include also the head due to the velocity of approach.

Equations for the discharge of water through pipes may be written as follows: Referring to Fig. 17, it may be seen that, $M = H + d$, and that, $H = z + F_z + I + F_t = Z + I + F = D + F$; then,

$$Q = \frac{6.301}{(d_1 + 4\, c\, l)^{0.5}}\, d_1^{2.5}\, (M - d_1)^{0.5} \quad \text{(94)}$$

General formulas may be written for weirs, as follows:

$$Q = 5.674 \left\{ \frac{3\, r}{3\, r + c\, l} \right\}^{1.5} b\, d^{1.5} \quad \text{(95)}$$

in which,

l = width of the crest of the weir;
c = coefficient of friction between water and masonry surface;
b = length of weir;
d = depth of water above crest of weir, when $l = 0$;
$Q = 3.089\ b\ m^{1.5}$;

or, when $l = 0$, Equation (95) becomes,

$$Q = 5.674\ b\ d^{1.5} \qquad (96)$$

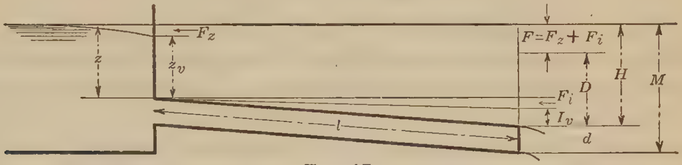


FIG. 17.

TABLE 15.—COMPARISON OF RIVER DISCHARGES.

Name of river. (1)	Approximate length of river, in miles. (2)	Catchment area, in square miles. (3)	Discharge, total maximum, in cubic feet per second. (4)	Discharge, in second-feet per square mile. (5)	COMPUTED VALUES FROM EQUATION (97), IN INCHES. d. (6)	M. (7)
India:						
Mahanadhi	520	45 000	1 570 000	35	1.27	2.54
Damodhar (above Raniganj)	200	7 200	650 000	90	1.28	2.56
Sonne	480	26 000	1 227 000	47	1.24	2.48
Ganges	1 500	368 000	1 800 000	5	1.20	2.40
Iravadhi	1 200	149 000	1 932 400	12.9	1.18	2.36
Godhaveri (Dowleshwaram)	600	115 870	1 510 000	13.2	1.24	2.48
Kistna	550	97 050	1 194 000	12.3	1.20	2.40
Cauvery	400	31 241	472 500	15	1.19	2.38
Indus	1 000	310 120	900 000	2.9	1.08	2.16
Jumna	860	138 180	1 333 000	9.6	1.28	2.56
America:						
Amazon	3 000	2 368 000	7 104 000	3.0	1.22	2.44
Mississippi	1 400	1 240 000	2 662 000	21.5	1.18	2.36
			2 250 000	26.2	1.22	2.44
Ohio (at Cairo, Ill.)	950	233 000	1 398 000	6.0	1.28	2.56
Genesee	22	2 428	54 000	22	1.23	2.46
Passaic	12	823	35 000	43.4	1.26	2.52
Niagara	35	263 000	298 000	1.13	1.49	2.98
Italy:						
Po	100	28 000	300 000	11.0	1.28	2.56
China:						
Yang-tse-Kiang	2 000	1 100 000	3 003 000	2.73	1.28	2.56
Austria:						
Danube (Isaktcha)	900	326 700	1 000 000	3.1	1.20	2.40
Spain:						
Maximum discharges of the Quadalquivir River:						
Menjibar*	80†	5 875	109 000	18.2	1.12	2.24
Cordoba*	120†	9 239	104 000	11.3	1.04	2.08
Palmado Rio*	120†	12 127	197 200	16.3	1.16	2.32
Cantidana*	120†	17 020	227 300	13.5	1.18	2.36

* Site of gauge.

† Maximum width of catchment basin, in miles.

Discharges from Catchment Basins.—Perhaps an extension of this principle of critical discharges would facilitate the calculation of discharges in rivers. If d in. represents the maximum average concentration of rainfall or surface flow of water into the river or tributaries at any time and b ft. is the total length of the rivers and tributaries, the maximum discharge of the river may be (roughly) deduced from Equations (95) and (96). If d is in inches and b ft. $= 5\,280\,B$ miles:

$$Q = \frac{5.671\ b\ d^{1.5}}{12^{1.5}};\ \text{or}\ 720\ b\ d^{1.5} \ldots\ldots\ldots\ldots\ldots\ldots (97)$$

Table 15 illustrates the usefulness of Equation (97).

The values given in Column (3) Table 15, are approximate and may have to be verified and corrected whenever necessary. The length was measured to the point where the rate of discharge was measured.

In the case of the Sonne River, the maximum mean intensity of rainfall in the basin contributing to the maximum peak on August 19, 1923, was 2.7 in.

The value of d or M would seem to depend on the intensity and duration of rainfall or to the depth of concentration of flow above ground at the river course, and the topography of the catchment basin; but the latter being a constant factor, it is probably only necessary to observe what relation actually exists between d or M and the intensity of rainfall. When that is determined the flood discharge in a river can be easily computed.

MELVIN D. CASLER,* M. AM. SOC. C. E. (by letter).—The discussions brought forth by this paper are all very helpful in focusing attention on the fact that the consideration of "stream flow in general terms" means much more than the simple appearance of the Chezy formula would indicate.

Mr. Bailey makes the very pertinent observation that it is often of greatest importance to appreciate that a given quantity of water with a given energy content, $d + h$, can flow at two different depths in a given channel.

He refers to the case of an entering or discharging stream at an intersection with a branch channel. In this case, as regards sections above and below the intersection, Q_x and Q_y are unequal, and the following treatment is suggested:

Let Q_x' and Q_x'' represent the respective flows of two entering streams; Q_y' and Q_y'', the respective flows of two departing streams; and, Q_x or Q_y, as the case may be, the flow at the junction. Other factors at Sections x and y are similarly indicated. I_1 and F_1 denote invert drop and total head loss, respectively, between the "primed" sections and the stream junction, while I_2 and F_2 denote the same between "double-primed" sections and the junction. Then, in the case of converging flow, $Q_x' + Q_x'' = Q_y$, and for diverging flow, $Q_x = Q_y' + Q_y''$. Following the reasoning used in the analysis of Fig. 1:

For converging flow,

$$W_x'\,(d_x' + I_1) + W_x'\,h_x' + W_x''\,(d_x'' + I_2) + W_x''\,h_x'' = W_y\,(d_y + h_y) + W_x'\,F_1 + W_x''\,F_2$$

Replacing each W by its corresponding $w\,Q$:

$$(d_y + h_y) = \frac{Q_x'}{Q_y}\,(d_x' + h_x' + I_1 - F_1) + \frac{Q_x''}{Q_y}\,(d_x'' + h_x'' + I_2 - F_2)$$

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* Mount Vernon, N. Y.

in which,

$$Q_y = Q_x' + Q_x''$$

Likewise, for diverging flow,

$$(d_x + h_x) = \frac{Q_y'}{Q_x}(d_y' + h_y' + F_1 - I_1) + \frac{Q_y''}{Q_x}(d_y'' + h_y'' + F_2 - I_2)$$

in which,

$$Q_x = Q_y' + Q_y''$$

Additional terms of like form may be appended to these equations to provide for three or more tributaries or branches with a common junction.

As stated by Mr. Pearce the paper presents nothing basically new. Its prime purpose is to facilitate the analysis of stream flow in irregular channels and to present a direct method of treatment applicable to all conditions, capable of being understood and applied by the average computer, and sufficiently short to permit the inclusion of certain generally accepted principles that are usually ignored because of the laborious calculations involved in their application by current methods—most of which sacrifice accuracy for simplicity, or *vice versa*. Most existing literature on this subject is either too vague in its generalities or else it is directly applicable only to certain special cases.

As suggested by Mr. Mavis, unless the significance of the friction factor, f, is clearly recognized there may be danger of confusing the writer's f with the factor, f, in the familiar pipe formula, $h' = f \frac{l\, v^2}{d\, 2\, g}$, in which, h' is the head loss in the length, l, due to perimeter friction, and d is the diameter of the pipe. In the writer's nomenclature, $h' = f\, l$, and, therefore, the writer's f equals the f of the pipe formula multiplied by $\frac{h}{d}$, in which, h is the velocity head, $\frac{v^2}{2\, g}$, and d is the pipe diameter.

With reference to Mr. Mavis' subsequent discussion it must be borne in mind that in the basic Equations (10) and (11), F is the total aggregate head loss between Sections x and y, due to all causes of whatever nature. The factor, f, is the unit head loss, due to perimeter friction only, and $F = \int_0^l f\, d\, l + F'$, in which, F' is the total head loss between x and y due to internal friction (eddies and impact due to sudden internal changes in velocity and direction). Equations (10) and (11), as stated, are theoretically correct and comprehensive and are applicable alike to one, two, or three-dimensional flows as long as Q is constant; that is, if no water enters or leaves the stream between Sections x and y.

The great obstacle in the way of the direct application of Equations (10) and (11) is the evaluation of the internal losses, F', which must include the losses incident to two or three-dimensional flows. The factor, F', is "the goat", so to speak, and must carry losses of sufficient aggregate magnitude to bridge all apparent discrepancies between Equations (10) and (11) and any and all observed hydraulic phenomena and experimental deductions.

At a hydraulic jump, Sections x and y are coincident and are operating at low stage and high stage, respectively; $l = 0$, $I = 0$, $F = F'$ = the head loss due to the jump, and, from Equation (10), $(d_x + h_x) - (d_y + h_y) = F'$; which refutes Mr. Mavis' suggestion that Criterion (3) would lead to the impression that $(d + h)$ is the same before and after the jump. Mr. Mavis overlooks the words, "as nearly as possible", in the criterion.

In the illustrative example plotted in Fig. 6, F' was ignored and, in the treatment of weirs, the entire, F, between Sections x and y was assumed as zero.

It is interesting to note that in a problem like that of Fig. 6, in which the channel is of constant cross-section, analyzed in stretches of uniform length for perimeter friction, or for any losses directly proportional to l, it is possible to eliminate all "cut and try" features by the use of a double curve as illustrated in Fig. 18. Assuming that $F = l$ times the average f between Sections x and y:

$$F = \frac{l}{2}(f_x + f_y) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (98)$$

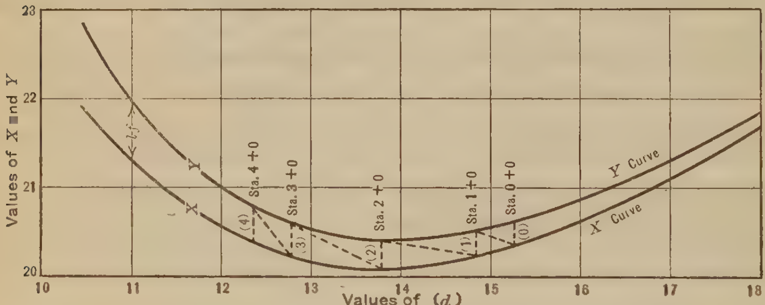

FIG. 18.

and, from the basic Equation (6),

$$d_x + h_x + I = d_y + h_y + \frac{l}{2} f_x + \frac{l}{2} f_y \ldots\ldots\ldots\ldots (99)$$

$$d_x + h_x - \frac{l}{2} f_x = d_y + h_y + \frac{l}{2} f_y - I \ldots\ldots\ldots\ldots (100)$$

$$d_y + h_y + \frac{l}{2} f_y = d_x + h_x - \frac{l}{2} f_x + I \ldots\ldots\ldots\ldots (101)$$

Let $X = d_x + h_x - \frac{l}{2} f_x$, and, $Y = d_y + h_y + \frac{l}{2} f_y$. Then, if proceeding up stream,

$$X = Y - I \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (102)$$

and, if proceeding down stream,

$$Y = X + I \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (103)$$

The problem of Fig. 6 may now be analyzed as illustrated in Table 16.

These values of X and Y are then plotted as shown in Fig. 18. From Table 5, the value of d at Station 2 + 0 (which is the critical section (see

Fig. 6)), is 13.8. From Fig. 18, for $d = 13.8$, X and Y are 20.07 and 20.44, respectively.

Proceeding up stream, the next value of X from Equation (102) is $20.44 - 0.20 = 20.24$, from which, $d = 14.85$ as plotted at (1) in Fig. 18, and Y for the next up-stream stretch $= 20.55$. The next value of X from Equation (102) is $20.55 - 0.20 = 20.35$, and $d = 15.3$ as plotted at (0) in Fig. 18.

TABLE 16.—HYDRAULIC ELEMENTS.

($Q = 6\,000$ sec-ft. $l = 100$ ft.)

d.	$h = m^* Q^2$.	$f = n^* Q^2$.	$X = d + h - \frac{l}{2} f$.	$Y = d + h + \frac{l}{2} f$.
10.469	11.866	0.0078	21.95	22.73
11.625	9.392	0.0058	20.73	21.31
15.000	5.422	0.0030	20.27	20.57
18.000	3.787	0.0019	21.69	21.88

* From Table 4.

Proceeding down stream from the critical section, the first Y from Equation (103) is $20.07 + 0.60 = 20.67$; then, $d = 12.8$ as plotted at (3) in Fig. 18 and X for the next down-stream stretch $= 20.22$. The next value of Y from Equation (103) is $20.22 + 0.60 = 20.82$, and $d = 12.4$ as plotted at (4) in Fig. 18.

The dotted zigzag line in Fig. 18 thus furnishes a graphical picture and a check of the analysis presented in Fig. 6 and Table 5.

It may be pertinent to observe that while the f curves of Figs. 2 and 4 only take care of that part of F due to perimeter friction, nevertheless the Q-curves of those diagrams are comprehensive in their application to the solution of Equations (10) and (11) and are just as applicable to the consideration of internal losses or combined losses as they are to the f losses.

Fig. 6 is a case similar to the convex invert slope referred to by Mr. Mavis. Whether the water is accelerated or decelerated below the break in grade may be determined by the proper application of Equations (10) and (11).

Regarding Mr. Mavis' reference to the impulse and momentum equations it may not be over sanguine to entertain the hope that there may be found ultimately a workable method of introducing those equations in conjunction with the energy equations in such a way as to relieve the internal friction factor, F', from some of the responsibility which it bears in Equations (10) and (11).

Mr. Mavis states in his reference to Criterion (4), that one would infer that a critical stage must be found before the calculations can be made. Such an inference was not intended. The computations may start from a critical section or from any other section where there is a condition of known depth for a known flow.

It should be noted that Criterion (3) refers only to the relations between two successive sections, one of which has already been determined, and that

Criterion (4) is merely a re-statement of Criterion (3) as applied to an analysis starting from the critical section. In the general criterion that F will be as nearly as possible equal to I, it is important to remember that F must include the losses due to any sudden changes in stage of flow.

An analysis is often started with an assumed depth at one end of the channel. If, at any stage in the progressive application of Equation (10) or Equation (11), a section is encountered at which the computed value of $(d + h)$ is less than the critical $(d + h)$ of that section for the assumed Q, that section is established as the critical section as regards all sections previously computed, and a new start is made at that section, revising all previous computations in reverse order.

If a known quantity is flowing at a known depth a comparison of the known d with the critical d as obtained from Equation (18) will establish the fact whether the channel at the point of known depth is operating at high stage or low stage. It may be pertinent to note that the depth of flow cannot pass from high stage to low stage, or *vice versa,* without passing through a critical section. If the transition is sudden a hydraulic jump or bore occurs, with attendant internal losses.

If, as suggested by Mr. O'Brien, there are simple, accurate methods of analysis available other than those presented by the writer, there is certainly indicated a dire need for a more general dissemination of such knowledge with illustrative examples of actual applications. All over the civilized world hydraulic engineers are daily ignoring the basic principles by applying simple approximations in the belief that more accurate analyses are impractical; or else they are laboriously plodding through reams of "cut and try" computations similar to those cited by Mr. Pearce, who demonstrates the speed and accuracy of the method of attack proposed by the writer. The only irksome feature of the writer's method is the preparation of the curves of Figs. 2 and 4 for the various sections involved. The computation of these curves and their critical factors may be readily standardized, as indicated in Tables 1, 2, 3, and 4, and reduced to systematic treatment. A computing machine is, of course, a great aid in any such operations.

Mr. O'Brien calls attention to the erroneous assumption of uniform velocity throughout the cross-section of a stream approaching a weir. This is a real error and is present in nearly all stream-flow calculations. The assumption is as stated in the definition of h in the "Nomenclature" and is again referred to in the parenthetical note following Equation (13) and the errors thus introduced are recognized and discussed in three places throughout the paper.* The profession would owe a vote of thanks to any one who presents a practical method of taking due account of non-uniform velocities. If their effect could be evaluated there would be some hope for the universal weir diagram presented in Fig. 8.

Mr. Simham sounds a warning against the application of the results of small laboratory experiments to practical cases of large magnitude, without due consideration of the general laws involved.

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* See pp. 19–20, 22–23.

Practical experimentation is an invaluable aid in establishing coefficients and often calls attention to certain physical laws and limitations and to various practical considerations which would never have occurred to the mathematician in concocting his formulas on a purely academic basis. Nevertheless, before attempting to make general empirical application of deductions drawn from laboratory experiments, every effort should first be made to interpret fully the experiments and deduce, if possible, the basic laws operating to produce the observed results and phenomena.

The quantity, Kpv^3, referred to by Mr. Simham, is the power loss (or energy loss per second) per running foot of channel; although the writer must admit that this fact may not be apparent from the paper.

The writer's use of W, or Q, as the water unit in his derivation of the Chezy formula, Equation (1), leads to some confusion, since Q is an appreciable quantity of water spread over an appreciable length of channel throughout which v cannot remain constant under general conditions. The results are the same no matter to what quantity of water the analysis is applied, but the significance of the various factors may be more apparent from the following derivation: Let q represent a minute elemental quantity of water at any particular cross-section of the channel, expressed in cubic feet. The factor, f, is the rate of head loss per foot of channel due to perimeter friction; and hence the energy lost by the quantity of water, q, per foot of forward movement is $w\,q\,f$ ft-lb. (Criterion 1).

The length of channel occupied by the quantity, q, is $\frac{q}{a}$, and the area of contact between q and the interior of the channel is $\frac{p\,q}{a}$ sq. ft.

The unit frictional resistance, in pounds per square foot of water contact, is taken as Kv^2, K being a constant and v being assumed as constant for the infinitesimal channel length, $\frac{q}{a}$, occupied by the quantity, q.

Hence, the total frictional resistance acting against the flow of the quantity of water, q, is $\frac{p\,q}{a}\,Kv^2$ lb.; and the work done by q per ft. of forward movement in overcoming this resistance is $\frac{p\,q}{a}\,Kv^2$ ft-lb. (Criterion 2).

Equating the work done to energy consumed, Criteria (1) and (2) furnish the following:

$$\frac{p\,q}{a}\,K\,v^2 = w\,q\,f \quad \text{.......................} \quad (104)$$

which, when simplified, becomes identical to Equation (1).

Since f is the head loss per linear foot of channel, $w\,f$ is the energy, in foot-pounds, lost by 1 cu. ft. of water in advancing over 1 lin. ft. of channel; and, since Q = the quantity of water passing, in cubic foot per second, the rate of energy loss at any particular cross-section of the stream, in foot-pounds per second per linear foot of channel, is $Q\,w\,f = w\,a\,v\,f$ (assuming the factor, f, to cover all losses).

From Equation (1), $w\,a\,v\,f = K\,p\,v^3$. Therefore, $K\,p\,v^3$, or its equivalent, $\frac{w\,p\,v^3}{C^2}$, is the power loss per linear foot of channel.

The ratio of the power-loss rate to the kinetic power delivered is seen to be $\frac{f}{h} = \frac{n}{m} = \frac{2\,g}{C^2\,r}$.

It would appear that Mr. Simham's equations are incomplete in that they take no account of the progressive changes in the hydraulic elements of the stream section.

In Equation (71) it is necessary to recognize that the quantities, p and v, represent coincident factors at one particular cross-section of the stream; that is, $K\,p_x\,v_x{}^3$ is the rate of power loss at Section x and $K\,p_y\,v_y{}^3$ is the rate of power loss at Section y. Hence, the total power loss between Sections x and y is,

$$\frac{K\,l}{2}\,(p_x\,v_x{}^3 + p_y\,v_y{}^3) \qquad (105)$$

It is also apparent that the total loss of power between Sections x and y is:

$$W\,F = \frac{W\,l}{2}\,(f_x + f_y) \qquad (106)$$

From Equations (105) and (106),

$$K\,(p_x\,v_x{}^3 + p_y\,v_y{}^3) = W\,(f_x + f_y) \qquad (107)$$

This reduces to

$$p_x\,v_x{}^3 + p_y\,v_y{}^3 = Q\,C^2\,(f_x + f_y)$$

Equation (107) is the true power-loss equation rather than Equation (71), and hence the resulting Equation (76) is, as Mr. Simham states, incomplete.

With reference to Mr. Simham's observations on the scouring of stream beds, the velocity of the water in immediate contact with the wetted perimeter is the determining factor in scouring, rather than the mean velocity; so that one of the factors affecting scouring is the degree of velocity uniformity in the cross-section of the stream. This consideration would lead to the conclusion that the mean velocity required to produce scouring of a given material depends largely on the shape of the stream section and on local conditions and circumstances.

A consideration of the general principles governing weir discharge brings up many interesting digressions, one of which is discussed by Mr. Simham under the head of "Discharges from Catchment Basins".

The writer would call attention also to Equation (39), in which, H_0 is the elevation of a reservoir flow line above the crest of a spillway weir, Q is the coincident flow over the weir, and b is the net width of the stream on the weir crest.

Reservoir flow-line elevations are commonly computed from the Francis weir formula, $Q = c\,b\,H_0{}^{\frac{3}{2}}$, with the assumption that $c = 3.33$. This reduces to,

$$H_0 = 0.4484\left(\frac{Q}{b}\right)^{\frac{2}{3}} \qquad (108)$$

The coefficient in Equation (39) is seen to be considerably greater than that in Equation (108). Furthermore, Equation (39) is predicated on zero friction losses in approaching the weir, and if appreciable friction losses exist the coefficient in Equation (39) would be even greater than 0.4717.

The trouble with Equation (108) is that its coefficient is obtained experimentally from measurements of H "a short distance up stream" from the weir, where the velocity of approach is assumed as being zero but where, as a matter of fact, there is undoubtedly some surface velocity even if $\frac{Q}{a}$ is negligibly small. The effects of these surface velocities are apparently not entirely eradicated even by the use of a stilling-box. Equation (38) would indicate that the value of c in the Francis formula, corresponding to an absolute zero velocity of approach, is 3.087.

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Paper No. 1723

FOUNDATIONS AND DRAINAGE OF HIGHWAYS*

By Albert C. Rose,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Harry Tucker, Winslow H. Herschel, Charles Terzaghi, D. P. Krynine, and Albert C. Rose.

Synopsis

This paper is based on field studies made by the writer in the Pacific Northwest during 1920 and 1921. The object of the studies was the development of simple and practical field tests and methods of identifying good, fair, and bad sub-grade soils.

As a result of the investigation the volumetric shrinkage of the soil was concluded to be the fundamental adverse factor. For measuring this, a linear shrinkage test was devised, as the displacement of pavements seemed to be caused principally by the distortion of the sub-grade in one direction—vertically. The procedure for the field moisture equivalent test was devised to prepare soils for the linear shrinkage test by wetting them with comparable quantities of water; but later it came to have a value of its own.

These tests were used on existing sub-grade soils, and the corresponding pavement conditions were observed. It was noted that an increase in the clay content of the soil, which was indicated by the test values, was accompanied by a corresponding adverse condition of the pavement.

Taken together, the field observations led to the conclusion that certain test limits could be used to identify good, fair, and bad sub-grade soils for the purpose of making a detailed sub-grade survey. A trilinear soil-classification chart, used in combination with a United States Bureau of Soils map,

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† Highway Engr., Bureau of Public Roads, U. S. Dept. of Agriculture, Washington, D. C.

further simplified the work and made possible reconnaissance sub-grade surveys without detailed tests.

The linear shrinkage and field moisture equivalent percentages were used to determine the character of the sub-grade soil as a foundation. The stability ratio was used to measure the drainage conditions.

INTRODUCTION

The proper design and construction of highway foundations and the methods of providing adequate drainage are research problems toward the solution of which the up-to-date highway engineer is devoting an intensive effort. The isolation of the sub-grade variable within recent years has accelerated the study of this fundamental road problem—the foundation. Intimately associated with the study of the character of the foundation is the kind of surface that is appropriate for a particular location.

The study of highway foundations and of drainage in connection with the surfaces involves two phases: (1) The economic; and (2), the engineering. The first determines the broad general policies, which establish the location of the road between controlling points; the amount of public revenues available for the work; the existing traffic requirements; a forecast of future development; the cost of operation of vehicles over various types of surfaces; and kindred subjects. The second, the engineering phase, although no less important, is more limited in its scope. It prescribes the design of grades and alignment to suit the terrain; measures the drainage requirements; plans the cross-section and surface of the road; and takes care of all the work involved in the design, construction, and maintenance of the roadway.

Only the engineering features of the problem will be discussed in this paper, and they will be limited principally to showing methods of detecting good and bad sub-grades and drainage, and to outlining the problem of designing surfaces. A careful analysis of engineering literature indicates that pavement, or surfacing, design involves four major variables: (1) The character of materials in, and the quality of workmanship on, the surfacing; (2) the local climatic conditions; (3) the character and condition of the sub-grade soil; and (4) the kind and amount of traffic. Probably at some future time these variables may be evaluated within broad limits, so that the highway engineer may be able to determine a rational surfacing design for any section of road when the local values of the variables are known.

Of the four variables mentioned, the sub-grade seems to be the most important; and in the sub-grade the volume changes caused by variations in moisture content and frost action are the factors that seem to be of greatest concern to the highway engineer. It is granted that no road surface would go to pieces without heavy traffic, but the distortion of the sub-grade due to volumetric changes must first take away the support from the surfacing before a pavement will go to pieces. As far as the other two variables are concerned, either they may be controlled, as by proper workmanship or material, or allowance may be made in a design for the known climatic conditions.

VERTICAL DEFORMATION THE FUNDAMENTAL ADVERSE CHARACTERISTIC OF SUB-GRADE SOILS

Analysis of the California Study.—In a paper,* C. A. Hogentogler, Assoc. M. Am. Soc. C. E., presents an enlightening and constructive analysis of the relative effect of various types of sub-grade soils on pavement conditions covering about 1 200 miles of concrete surface in the California State Highway System in 1920. His tabulated analysis (Table 1) indicates that the undesirable conditions of the sub-grade soils increase with the clay content. On this basis, soils may be classified with regard to their influence on pavement condition.

TABLE 1.—PAVEMENTS CLASSED AS DESIRABLE OR UNDESIRABLE, USING ONLY THREE SUB-GRADE CLASSIFICATIONS.

Type of sub-grade.	PAVEMENT CONDITION.	
	Desirable percentage.	Undesirable percentage.
Sand	100.0	....
Loam	93.0	7.0
Clay	79.6	20.4

These same general conclusions were advanced by the writer after making a number of field studies of sub-grade soils as to their effect on pavements in the States of Oregon and Washington during 1920 and 1921. These investigations were prosecuted with equal diligence at all seasons of the year so that the results represent the extremes as well as average conditions of the sub-grade soils as affected by climatic variations.

Development of Field Moisture Equivalent and Linear Shrinkage Tests.—At the time these field studies were initiated the writer discussed the problem with highway engineers and agricultural soil experts, consulted soil bibliographies and literature, and made reconnaissance surveys of existing field conditions. Based on this preliminary information the indications were that the volumetric change in the sub-grade soils was the fundamental factor causing cracking, displacement, and other unfavorable conditions of pavements. This volumetric change might be caused either by variations in the moisture content of the soil or by frost action. It was evident, therefore, that some test should be devised for measuring the comparable volumetric changes between soils varying in the size, as well as in the character, of the component grains. It was further evident that this test should measure the linear shrinkage rather than the volumetric shrinkage of the soil since the distortion of the sub-grade that caused pavement displacement acted mainly in one direction—vertically.

The standard moisture equivalent test seemed to be the most promising method for wetting the soils with comparable quantities of water prior to making shrinkage tests.* It was considered probable that, in general, a force 1 000 times that of gravity would leave a surface film of water of equal thickness around all soil grains even if they varied in size and character. This would depend, of course, on whether the individual grains were porous or solid, or had glazed or roughened surfaces.

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* "California Road Survey Demonstrates the Economic Possibilities of Subgrade Studies," by C. A. Hogentogler, *Public Roads,* Vol. 7, No. 12, February, 1927.

An examination of the standard centrifuge for making the moisture equivalent test showed it to be a machine designed for the laboratory and unsuitable for the general field use of the highway engineer. It was decided, therefore, to duplicate if possible, with an inexpensive but simple and practical field test, the results secured with the standard apparatus for determining the moisture equivalent.

The quantity of water absorbed by a soil due to surface tension and adhesion was compared with the water content retained by the soil against a force equivalent to 1 000 times that of gravity (the standard centrifuge force). To do this the soil was powdered to pass a 1-mm. screen and water was dropped from a burette on the sample in an iron bowl with constant stirring. When the material attained the consistency of putty, it was compacted and the water was allowed to drop on the surface as long as it was absorbed. When the absorption was complete the surface assumed a wet, shiny appearance, which indicated the critical point of the test. Fig. 1 illustrates three stages in the test.

The water content of the soil divided by the dry weight of the soil and the result multiplied by 100 gave the "field moisture equivalent". It was found that this percentage could be determined accurately for soils if it were 20, or more. Further field investigation showed that where soils gave lower values they were so coarse-grained as to drain readily, and, therefore, were practically always good sub-grade soils. Consequently, extreme accuracy for soils with a field moisture equivalent of less than 20 was not considered of practical significance.

Comparison of Field Moisture Equivalent with Standard Moisture Equivalent.—An analysis by least squares of the field moisture equivalents for twenty-nine soil samples, with values ranging from 20 to 41%, indicated that they were slightly closer to the standard moisture equivalent results, on portions of the same samples centrifuged in the laboratory of the U. S. Bureau of Soils, than similar laboratory tests made by the Oregon Agricultural College.† The field moisture equivalent method was used, therefore, for a comparative wetting of various soil samples before they were moulded into 1 by 1 by 10-in. bars (Fig. 2) and shrunk to determine the linear shrinkage value. This was considered to be the difference between the wet and dry length of the bars computed as a percentage of the wet length.

Check runs of these two simple and practical field tests indicated that, while they did not give identical results they gave consistent values, as close as those obtained by other routine engineering tests. It was found that a general relation existed between the values of the field moisture equivalent and the linear shrinkage, and that for tentative investigations the simpler test (field moisture equivalent) was indicative of the results to be obtained by the more difficult test (linear shrinkage). For absolutely accurate work, however, the linear shrinkage was considered the critical test.

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* The making of shrinkage tests was suggested by C. L. McKesson, M. Am. Soc. C. E., now Materials and Research Engineer of the California State Highway Commission. The writer carried on the field studies for the Bureau under Mr. McKesson's immediate direction.

† "Practical Field Tests for Subgrade Soils," by A. C. Rose, *Public Roads*, Vol. 5, No. 6, August, 1924.

Later, it was observed that the field moisture equivalents of a large number of soils were numerically similar to the clay contents of the soil in percentage as determined by the mechanical analyses of the U. S. Bureau of Soils shown in its *Soil Survey Bulletins.*

Comparison of Pavement Conditions with Test Values

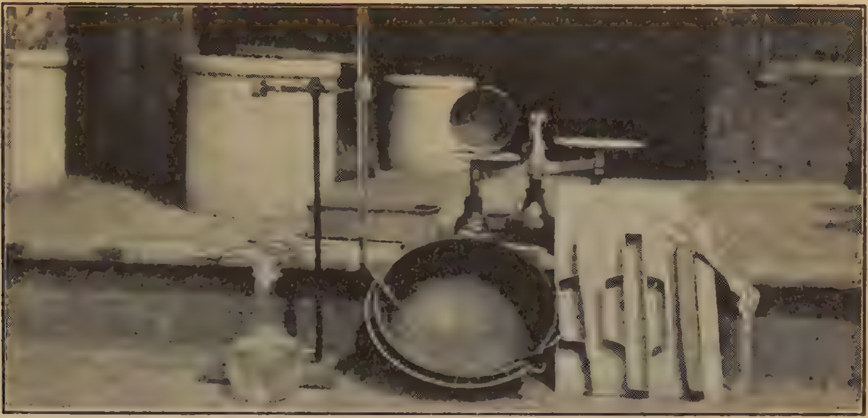
Subsequent to the development of these tests the writer made a number of field studies to determine what, if any, relation existed between the condition of the pavement and the test values of the sub-grade soil. Out of these investigations came the conclusion that the sub-grade soil was more important in its adverse effect on the pavement than the kind or amount of traffic. Pavements may crack as a result of temperature variations, but they are rarely broken up and disintegrated by traffic, except when on an unstable sub-grade or on one distorted by unequal volume changes in the sub-grade soil. Under the latter condition rigid pavements act as cantilevers or beams and are more readily broken, while non-rigid pavements become depressed, shattered, or broken through.

Sub-Grade Failure Under a Concrete Pavement.—One of the investigations covered the sub-grade failure under a concrete pavement, 2 miles long, between Centralia and Chehalis, Wash., laid during June, July, and August, 1914 and 1915. The severity of a part of the failure is illustrated by Figs. 3 and 4. The adverse humid climate of the region is shown by Fig. 5. During construction the average temperature was 65° Fahr. Since then the variation has been from a maximum of 99° Fahr. to a minimum of —16° Fahr., with a maximum contraction range of 81° Fahr. and a maximum expansion range of 34° Fahr.

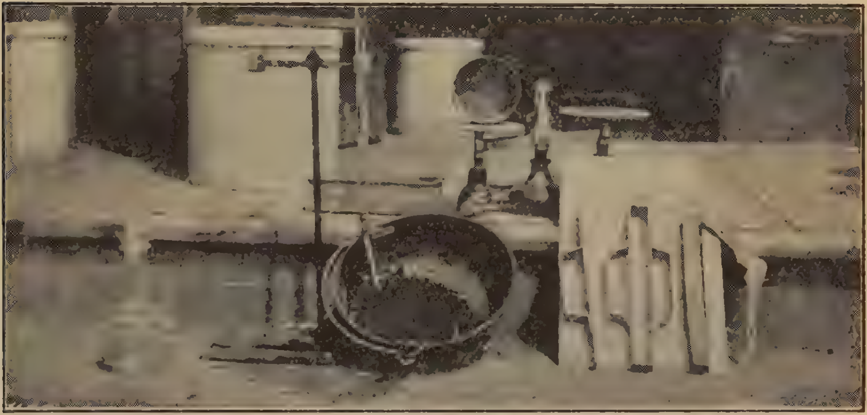
The pavement was divided into four sections based on the degree of failure in each portion. In each of these sections a 400-ft. length was selected as representative. The detailed tests were confined to these 400-ft. sections.

The investigation showed unmistakably that the degree of failure of the test sections corresponded with the increase in the values of the field moisture equivalents and linear shrinkages of the sub-grade soil. The latter are summarized in Table 2. The corresponding condition of the test sections is represented by crack charts (Fig. 6) and photographs (Figs. 7 to 10, inclusive). An adjacent section of concrete pavement laid on a sandy soil, with a linear shrinkage percentage of zero, was practically free from cracks or other defects.

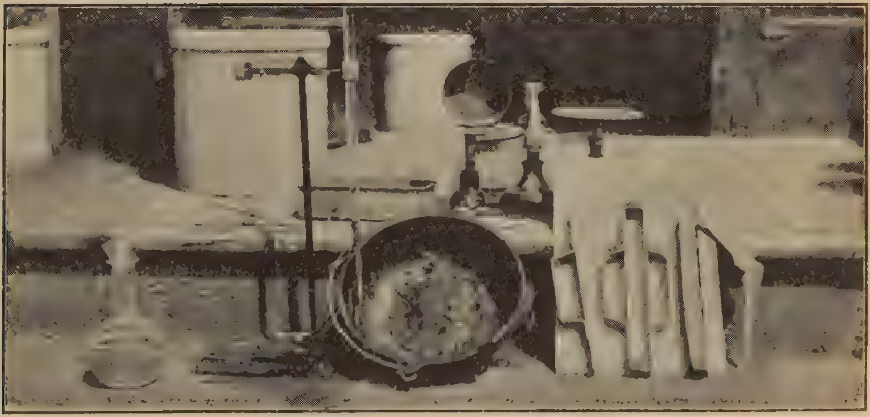
The failure of this pavement could not be attributed to defective concrete. Although tensile and compressive tests were not made, it was evident that the concrete was good. It was sound under the blow of a hammer and when broken showed a normal fracture. The construction inspector's records indicated that an adequate quantity of cement had been used for the specified mixture.



(*a*) Sample Dry and Pulverized.

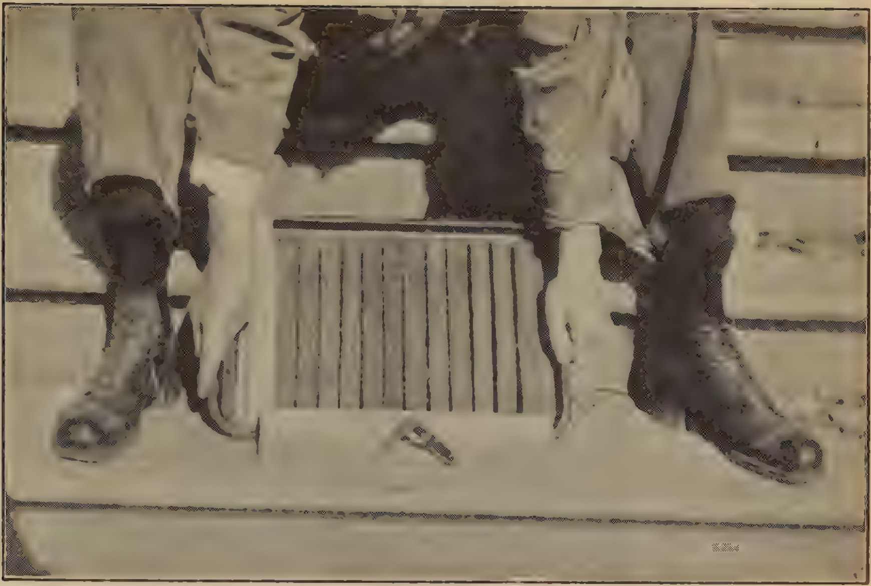


(*b*) Sample Wetted to One-Half the Field Moisture Equivalent Percentage.

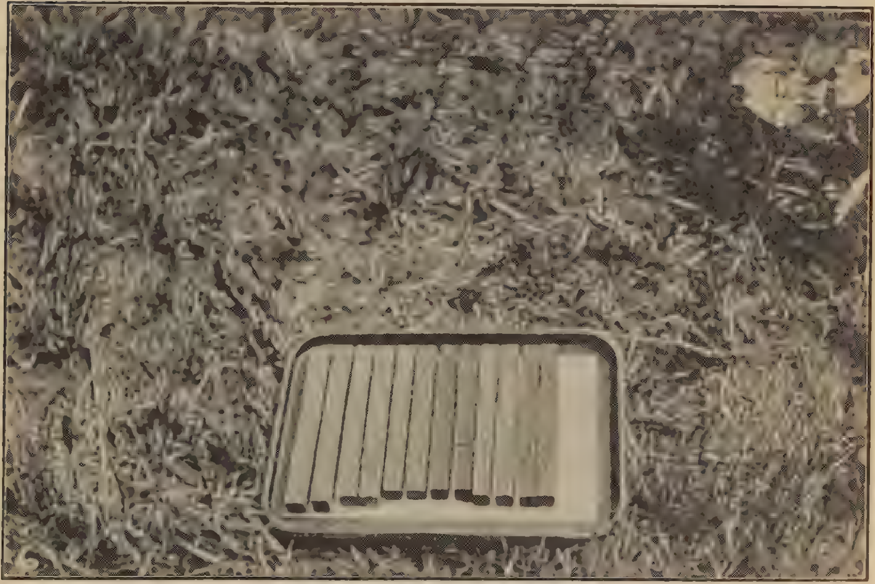


(*c*) Sample Wetted to the Total Moisture Equivalent Percentage.

Fig. 1.—Apparatus for Making Field Moisture Equivalent Test.



(*a*) Soil Bars Before Being Shrunk.



(*b*) Soil Bars After Being Shrunk.

Fig. 2.—Determination of Linear Shrinkage.

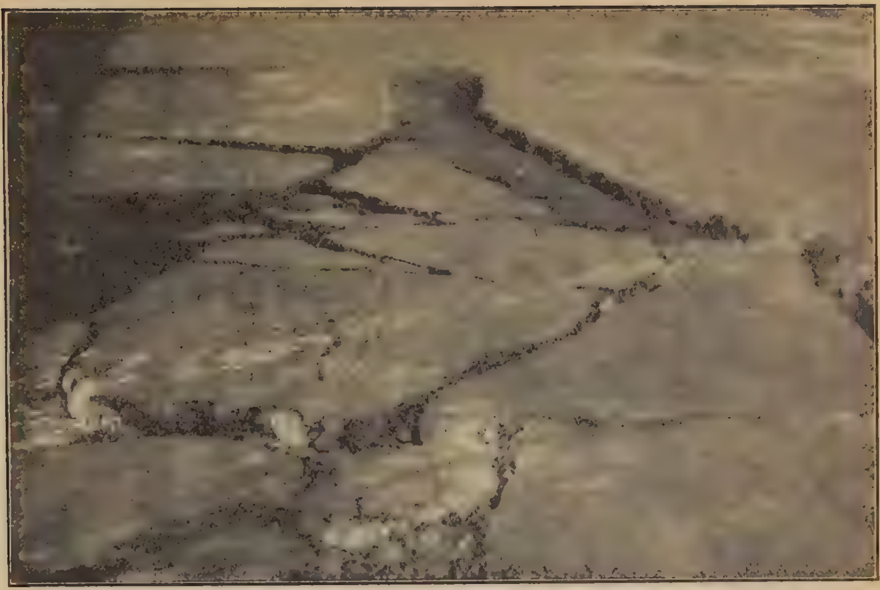


FIG. 3.—SEVERE CRACKING OF CONCRETE PAVEMENT ON CENTRALIA-CHEHALIS ROAD, WASHINGTON.

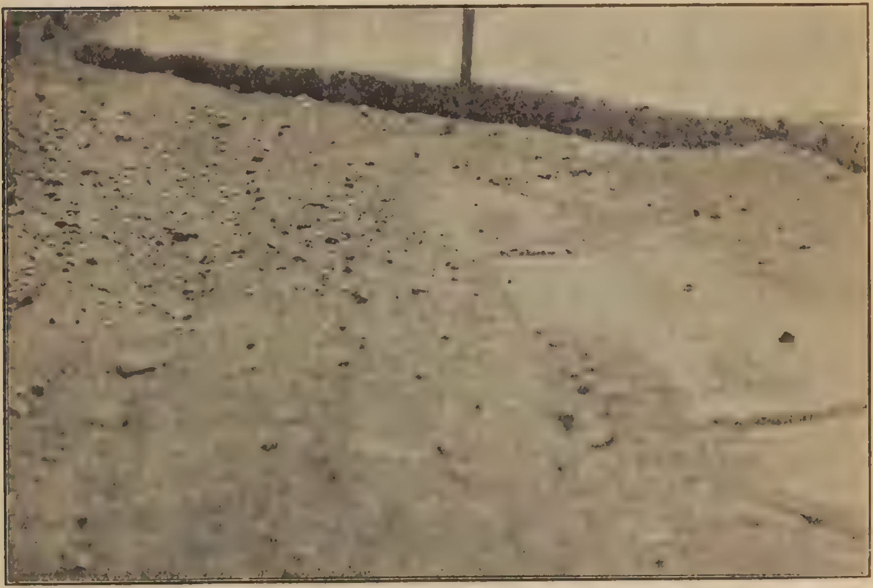


FIG. 4.—TYPE OF CONCRETE PAVEMENT FAILURES REPRESENTED BY TEST NO. 4, CENTRALIA-CHEHALIS ROAD, WASHINGTON.

The increase in the clay content of the soil corresponded with the degree of severity of the failure even where the sub-grade varied from cut to fill, and where the fill varied in height and was overflowed annually by the flood water of the adjacent river. The mechanical analyses of the soil types, as made by the U. S. Bureau of Soils, are shown in Table 3.

TABLE 2.—SUMMARY OF REPRESENTATIVE SOIL TEST VALUES ON TESTS NOS. 1 TO 4, INCLUSIVE.

Test number.	Section of pavement represented, station to station.	Relative condition of pavement in section.	California study classification.	Classification of soil by U. S. Bureau of Soils.	SOIL TEST VALUES.	
					Percentage of field moisture equivalent.	Percentage of linear shrinkage.
1	0 to 56	Best....	*C*	Chehalis clay loam.	30.8	2.1
2	56 to 65	Second best ..	*D*	Chehalis clay loam.	35.6	6.6
3	93 to 107 + 84	Third best....	*D*	Transition between Chehalis clay and clay loam	42.4	11.3
4	65 to 93	Worst	*E*	Chehalis clay	55.1	13.1

Sub-Grade Failure Under a Bituminous Pavement.—Another serious sub-grade failure was investigated under a Warrenite bituminous pavement, 4 852 ft. long, extending north from the southern city limits of Chehalis, Wash.

TABLE 3.—THE MECHANICAL ANALYSES OF SOIL TYPES ENCOUNTERED ON TESTS NOS. 1, 2, 3, AND 4, AS DETERMINED BY THE U. S. BUREAU OF SOILS.

Type.	Description.	Percentage of fine gravel.	Percentage of coarse sand.	Percentage of medium sand.	Percentage of fine sand.	Percentage of very fine sand.	Percentage of silt.	Percentage of clay.
Chehalis silty clay loam......	Soil.......	0.0	0.1	0.1	2.6	5.8	64.8	26.4
	Subsoil...	0.0	0.0	0.2	1.9	1.0	64.7	32.1
Chehalis clay loam	Soil.......	None listed.						
	Subsoil...	None listed.						
Chehalis clay..................	Soil.......	0.0	0.1	0.1	0.7	0.2	24.0	74.6
	Subsoil...	0.0	0.0	0.0	0.3	0.5	20.7	78.3

The pavement failures were classified as: (1) Depressed areas (D. A.); (2) incipient failures (I. F.); (3) shattered areas (S. A.); and (4) broken through areas (B. T.). Any depression that averaged more than 0.05 ft. below a 6-ft. straight-edge was considered a depressed area. Incipient failures were depressed areas or others in which the pavement had begun to fracture but no shattering or disintegration had yet occurred. The areas were measured until the observer became sufficiently familiar with the dimensions to estimate them. An automobile speedometer was used to locate the failures along the project. A sample of the form used is shown in Table 4.

The tests of the sub-grade soil showed the field moisture equivalents to vary from 26 to 45 per cent. From previous tests,* the corresponding linear shrinkages for these moisture equivalents were estimated at 3 to 9 per cent. The soil map made by the U. S. Bureau of Soils over this area showed the predominating soil types to be members of the Salkum series. The mechanical analyses in Table 5 show the clay content to be well in excess of 30 per cent.

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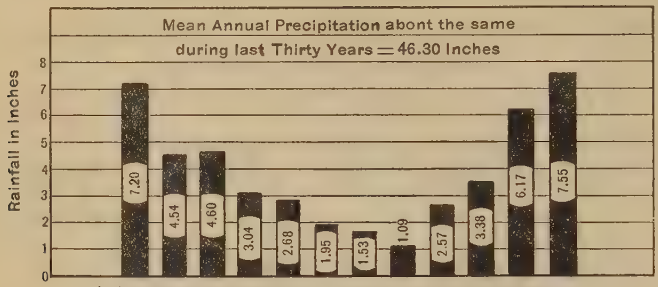
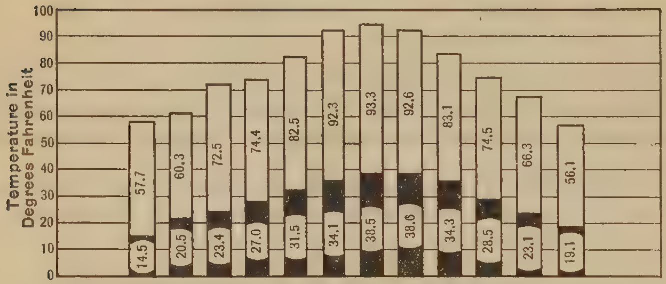
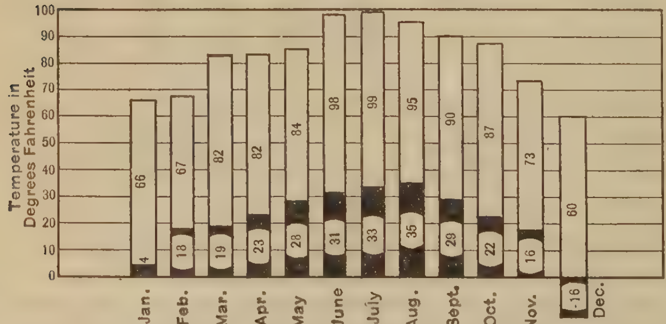
* "Practical Field Tests for Subgrade Soils", by A. C. Rose, *Public Roads*, Vol. 5, No. 6, August, 1924.



(a) AVERAGE MONTHLY PRECIPITATION 1911-1920



(b) AVERAGE MAXIMUM AND MINIMUM TEMPERATURES 1910-1920



(c) ACTUAL MAXIMUM AND MINIMUM TEMPERATURES SINCE 1914

Fig. 5.—Climatological Data for Region in Vicinity of Centralia and Chehalis, Wash., Compiled from Records of U. S. Weather Bureau.

The investigation indicated that the 2-in. Warrenite wearing surface, on a 2-in. new stone course, over a 6-in. old macadam road, was not heavy enough to withstand the traffic under the existing adverse soil conditions and the inadequate surface drainage and sub-drainage. As a whole, the pavement was a failure, 34.6% of the total area being defective, distributed throughout the entire length of the pavement.

Results of the Field Investigations in the Pacific Northwest.—The foregoing two examples illustrate the findings of the sub-grade studies in the Pacific Northwest. As a result the linear shrinkage of the soil was considered to be the critical test and the field moisture equivalent was taken as a simple and rapid indication of this value. The exceptions to the rule have been noted in previous papers by the writer.*

TABLE 4.—Sample of Form Used for Classifying Pavement Defects in Bituminous Pavement Represented by Tests Nos. 5 and 6.

Mile.	Kind of failure.	Dimensions, in Feet.		Distance from Edge of Pavement to Nearest Point of Failure, in Feet.		Area Affected, in Square Feet.	
		Length.	Width.	Left.	Right.	Left.	Right.
0.30	D. A.	3	10	5	......	30	......
0.48	D. A.	5	18	......	0	......	90
0.58	I. F.	2	2	7	......	4	......
0.70	S. A.	5	6	......	6	......	30
0.75	B. T.	18	150	0	0	1 350	1 350

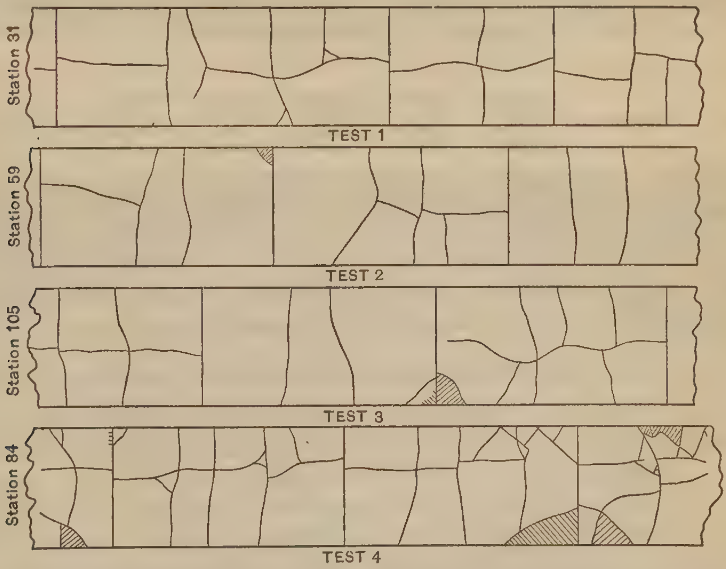


Fig. 6.—Typical Condition Charts of Concrete Pavement at the Several Test Sections.

The method was further simplified for rough reconnaissance purposes by considering the clay content of the soil (expressed in percentage) as indicative of the field moisture equivalent, and, therefore, of the shrinkage value of the soil, a relation that experience had shown to be reasonably safe. The advantage to be gained by using this relation lies in the fact that it enables the highway engineer to interpret, for purposes of sub-grade study, the great volume of data already made available by the U. S. Bureau of Soils, showing the mechanical analyses of soils of all varieties found in all sections of the country. For purposes of ready interpretation of these data a trilinear soil classification chart was developed (Fig. 11). The classification of sub-grade soils by texture is, as follows:

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* "Practical Field Tests for Subgrade Soils", *Public Roads*, Vol. 5, No. 6, August, 1924; "Field Methods Used for Making Subgrade Surveys," *Public Roads*, Vol. 6, No. 5, July, 1925; and "The Present Status of Subgrade Studies," *Public Roads*, Vol. 6, No. 7, September, 1925.

Soils containing less than 20% of clay:

- Coarse sand—more than 25% of fine gravel and coarse sand, and less than 50% of any other grade.
- Sand—more than 25% of fine gravel, coarse and medium sand, and less than 50% of fine sand.
- Fine sand—more than 25% of fine sand or less than 25% of fine gravel, coarse and medium sand.
- Very fine sand—more than 50% of very fine sand.

Soils containing 20 to 50% of silt and clay:

- Sandy loam—more than 25% of fine gravel, coarse and medium sand.
- Fine sandy loam—more than 50% of fine sand or less than 25% of fine gravel, coarse and medium sand.
- Very fine sandy loam—more than 35% of very fine sand.

TABLE 5.—MECHANICAL ANALYSIS OF SALKUM SILTY CLAY AS DETERMINED BY THE U. S. BUREAU OF SOILS.

Type.	Description.	Percentage of fine gravel.	Percentage of coarse gravel.	Percentage of medium sand.	Percentage of fine sand.	Percentage of very fine sand.	Percentage of silt.	Percentage of clay.
Salkum silty clay	Soil..........	0.5	1.2	1.1	3.3	3.1	59.4	31.5
	Subsoil......	0.2	1.1	1.2	3.2	3.6	55.6	34.6

The type names in the several areas of Fig. 11 agree with the nomenclature used by the U. S. Bureau of Soils in making its soil maps.

METHOD FOR MAKING SUB-GRADE SURVEYS DEVELOPED FROM FIELD STUDIES

Correlation of Test Values with Existing Soil Maps.—With this trilinear soil classification chart, the problem of making a reconnaissance sub-grade soil survey in areas mapped by the U. S. Bureau of Soils is greatly simplified. The maps show the location of the various soil types and the points on the roads at which the character of the soil changes. The *Soil Survey Bulletins* published by the Bureau contain mechanical analyses of all the soils shown on the maps. The nomenclature of the maps and the *Bulletins* agree with that of the trilinear soil classification chart.

It becomes a simple matter then for the engineer studying sub-grade conditions to determine from the maps the type of the soil at particular locations, and, by the use of the *Bulletins* and the trilinear chart and by the employment of the observed relation between the field moisture equivalent and clay content, to distinguish between the soils that are clearly good sub-grades and

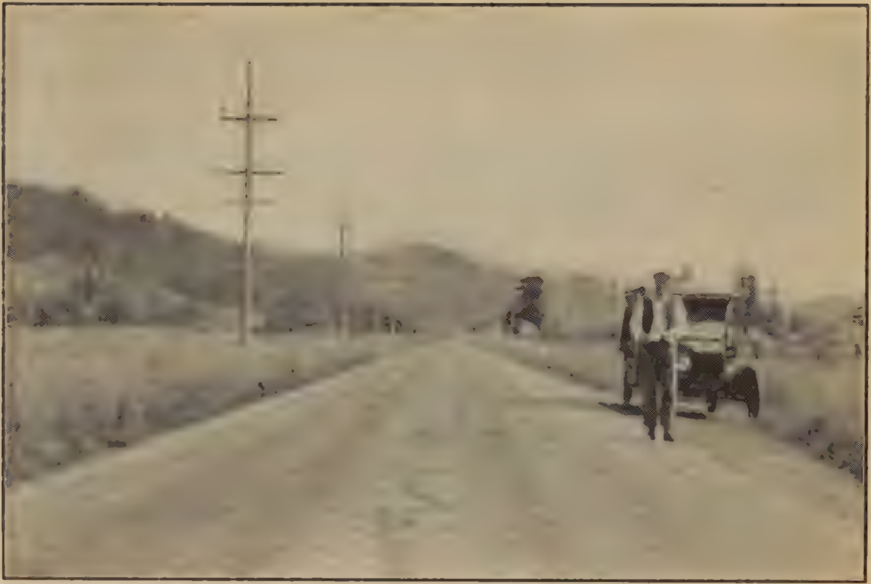

FIG. 7.—VIEW OF PAVEMENT IN TEST NO. 1.

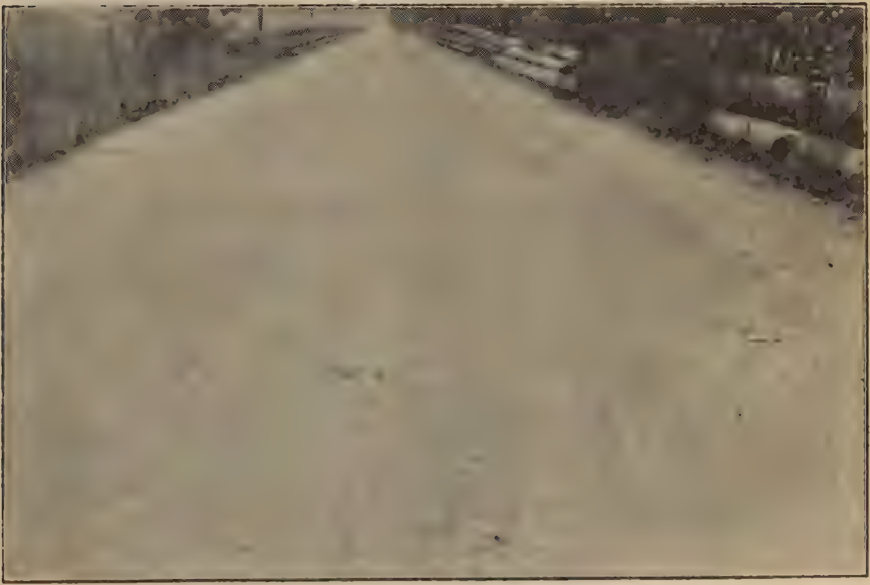

FIG. 8.—SECTION OF TEST NO. 2 PAVEMENT.

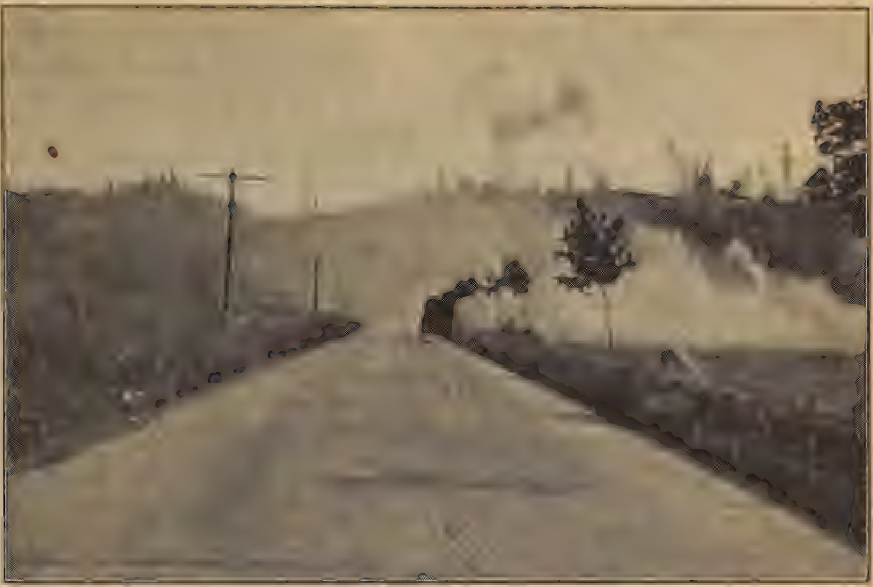

FIG 9.—TYPICAL VIEW ON TEST NO 3 PAVEMENT.

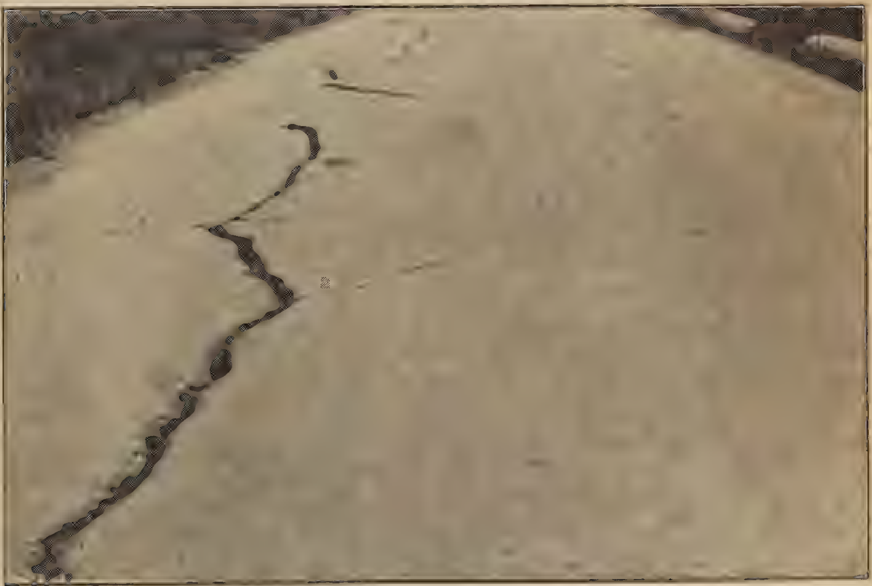

FIG. 10.—TEST NO. 4, REPRESENTATIVE SAMPLE OF PAVEMENT.

those that are clearly bad sub-grades. Of course, there will always be some soils of doubtful character, which it will be necessary to test either by the simplified methods described herein or by more elaborate laboratory methods; but the number of such cases is relatively small.

The field observations indicated that for the purpose of making sub-grade soil reconnaissance surveys, the clay content by mechanical analysis may be considered equal to the field moisture equivalent. When the field moisture equivalent was less than 20, the linear shrinkage percentage was usually negligible and the soil made a good sub-grade; when the field moisture equivalent varied from 20 to 30, the linear shrinkage percentage usually did not exceed 5 and the soil made a fair sub-grade; and when the field moisture equivalent exceeded 30, the linear shrinkage percentage usually was greater than 5% and the soil made a poor sub-grade.

Class	%Sand	%Silt	%Clay
Sand	80–100	0–20	0–20
Sandy loam	50-80	0-50	0-20
Loam	30–50	30–50	0-20
Silt loam	0–50	50–100	0-20
Sandy clay loam	50-80	0-30	20-30
Clay loam	20-50	20-50	20-30
Silty clay loam	0-30	50-80	20-30
Sandy clay	55-70	0-15	30-45
Clay	0-55	0-55	30-100
Silty clay	0-15	55-70	30-45

Size of Particles

Grade	Millimeter
Coarse material	more than 2
Fine gravel	2 to 1
Coarse sand	1 to 0.5
Medium sand	0.5 to 0.25
Fine sand	0.25 to 0.1
Very fine sand	0.1 to 0.05
Silt	0.05 to 0.005
Clay	0.005 to 0.0

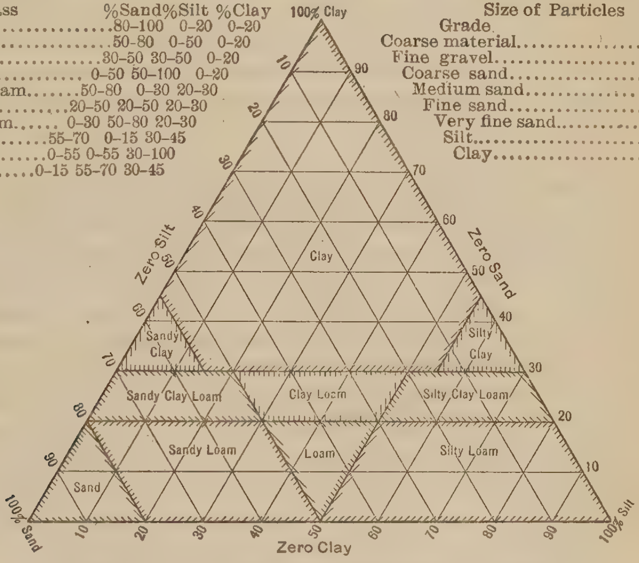

FIG. 11.—TRILINEAR SOIL CLASSIFICATION CHART.

The results vary considerably in individual cases and are based on observations made in the Pacific Northwest. Before they are accepted for any other region, or for any particular project, they should be verified by actual tests.

The tests for field moisture equivalent and linear shrinkage are not claimed to indicate good, doubtful, and bad sub-grade soils under all conditions. No tests, however refined, that have yet been devised, will accomplish this. In the field studies in the Pacific Northwest certain pavement conditions could not be explained on the basis of the field test values. Although not 100% effective, the tests, however, were significant, in the great majority of cases, and it is believed that they will be of value to the field engineer until some simpler and more effective method is devised.

They were also found to be usable by a highway engineer with only moderate preliminary training. They are now being used by the State of California and have been included in those under investigation by the U. S. Bureau of Public Roads. Frank H. Eno, M. Am. Soc. C. E., in his summary of the progress of soil investigations states* that "Rose's limits for shrinkage at 5 per cent. and moisture equivalent at 20 per cent. seem to be working out satisfactorily in practice where they have been used."

The writer believes that the results secured with the field tests developed in the Pacific Northwest and their subsequent use by other States have demonstrated their usefulness for the detection of good, doubtful, and bad subgrade soils. The consistency of the test results has been verified in the laboratory of the U. S. Bureau of Public Roads by the close agreement between check tests made by the same investigators and between those on the same samples made by different investigators. They are advocated primarily as field tests designed for simplicity and practicability. They give a direct determination of the fundamental adverse factor of the sub-grade soil—the volume change in one direction (the vertical), represented by the linear shrinkage percentage. In addition, test limits have been devised based on the values obtained by these tests, which are useful criteria for determining the relative character of sub-grade soils.

Sub-Grade Soil Classification Now in Progress.—The Bureau of Public Roads with the aid of its laboratory has recently initiated an elaborate investigation to determine the upper and lower test limits of the various soil series and types throughout the country, as identified by the nomenclature of the Bureau of Soils. This study extends over all the States—investigations that previously had been confined to individual States scattered throughout the West and East. These soil tests will be carried on in connection with studies of pavements in the various regions, the object being to correlate definite soil test values with the existing pavement conditions. In order to reduce the required laboratory testing to a minimum it is planned to group all subgrade soils tentatively on the basis of the values secured by the Atterberg plasticity test,† which is comparatively simple to perform and gives information on soil characteristics varying from low to high values.

Later, more elaborate tests will be made on representative samples of each of these groups, to determine the maximum and minimum values of such engineering characteristics as the compressibility, elasticity, and permeability. The values will be determined for each soil horizon (or depth below the surface) for each group, so that the highway engineer may know the effect of raising or lowering the grade line. By this method, in connection with correlated observations on existing road conditions, it is hoped to identify all the U. S. Bureau of Soils types and series with regard to their efficiency as sub-grade soils. Under this plan Melbourne clay might be found to fall within the doubtful group, while a Chehalis clay might always lie in the bad group.

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* "Progress in Subgrade Soil Investigations," by Frank H. Eno, Paper delivered before the Twenty-fourth Annual Convention of the Am. Roadbuilders Assoc., Chicago, Ill., January 13, 1927.

† "Simplified Soil Tests for Subgrades and Their Physical Significance," by Charles Terzaghi, M. Am. Soc. C. E., *Public Roads*, Vol. 7, No. 8, October, 1926.

Also, it might be found best to lay the grade line of the highway in the good surface horizon rather than on the bad sub-soil.

When this work is completed it is hoped that the soil series and types as mapped by the U. S. Bureau of Soils will be converted into terms that are usable for the highway engineer. The objective of all these laboratory tests and correlated investigations on road conditions will be to reduce the necessary testing to a minimum. Thus, for any given region, the soils will be classed with respect to their physical characteristics, and later, when field survey data have been received, they will be classed with regard to the physical condition of the pavements laid on them.

Until the results of these tests become available, however, and even after the work is virtually completed, there will always be a need for practical and inexpensive field tests to identify the character of the doubtful soils. It is to satisfy this need that the field tests and methods suggested in this paper have been formulated.

Definition of the Stability Ratio

In making the shrinkage tests it was noticed that the soil could not be compacted into solid bars when the moisture content exceeded the field moisture equivalent. This same plasticity was observed in making the field auger borings. When the moisture content was less than the field moisture equivalent, the drillings issued from the earth in hard, shiny "worms". When the moisture content was excessive, the condition of the soil varied from sloppy to putty-like, to soft, to friable; and the drill could be pushed down without rotation.

For these reasons the stability ratio of the soil was defined as the moisture content divided by the field moisture equivalent (both in percentage).* When the stability ratio of a sub-grade soil is greater than 1, the soil is considered to be in an unfavorable condition. This ratio applies only to the fine-grained soils with a field moisture equivalent in excess of 20. The coarse-grained soils drain readily and do not seem to be materially affected by a like moisture content.

The observations on well-drained soils in the Pacific Northwest in the open fields, or in highway rights of way not under a pavement surface, indicated that the maximum moisture contents of the soils were less than the field moisture equivalents. This applied to samples taken far enough beneath the surface (about 3 to 6 in.) to eliminate the effect of water standing in slight depressions or of sheets of run-off water. This condition was found to exist even in the rainy season and within 3 or 4 hours after heavy rainfalls.

On poorly drained soils, where surface depressions or restricted sub-drainage existed, the moisture content of the natural soils, at depths below the surface comparable with the well-drained soils, generally exceeded the field moisture equivalent.

In 1925, Professor F. J. Alway, of the University of Minnesota, stated that sufficient investigations had been made over the country at large to show that in well-drained soils the maximum moisture contents throughout the year usually were equal to or less than the moisture equivalent. It should be noted, however, that Professor Alway's statement referred to the standard moisture equivalent made against a force equivalent to 1 000 times that of gravity. Although the field moisture equivalent seems to approximate the standard moisture equivalent closely, it is not made against any force except the surface tension of the water about, and the adhesion to, the soil particles. Therefore, it should not be confused with the standard laboratory test.

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* "Practical Field Tests for Subgrade Soils," by A. C. Rose, *Public Roads,* Vol. 5, No. 6, August, 1924.

On observations under pavements, with few exceptions, the moisture contents of the soil at points of failure exceeded the field moisture equivalent. The converse was true for sections of pavement in good condition.

The field observations on natural soils not covered by pavements, as well as on sub-grade soils, indicate that there is a possibility by proper pavement design of controlling the moisture content in a sub-grade soil so that it will rarely exceed the field moisture equivalent.

SUMMARY

These studies of the engineering factors in highway foundations and drainage show definite prospects of ultimate benefit. In the future the variables may be evaluated within broad limits, so that it will be possible to determine an adequate pavement design for any section of road where these factors are known. The following conclusions may be drawn:

1.—The adverse character of the sub-grade soil depends primarily on the volume change caused by variations in moisture content or frost action. The main adverse action of the volume change occurs in one direction—the vertical.

2.—The volume changes of the soil distort the sub-grade, which displaces the pavement unevenly and subjects it to excessive localized stresses.

3.—The adverse character of the sub-grade soil is shown to increase with the clay content.

4.—The clay content and adverse character of the sub-grade soil is indicated by certain simple and practical field tests, called the field moisture equivalent and the linear shrinkage tests.

5.—Detailed sub-grade soil surveys may be readily made by using U. S. Bureau of Soils maps in connection with these field tests.

6.—A simplified and rapid method for making a reconnaissance soil survey is by the use of a U. S. Bureau of Soils map in connection with a trilinear soil classification chart, without the use of field or other tests. Generally speaking, sands, sandy loams, loams, and silty loams are considered good sub-grade soils; sandy clay loams, clay loams, silty clay loams, sandy clays, and silty clays, doubtful sub-grade soils; and clays, bad sub-grade soils.

7.—The drainage condition of a sub-grade soil, as well as the bearing power, is indicated as poor when the critical point—a stability ratio of one—is exceeded. The stability ratio is defined as the moisture content divided by the field moisture equivalent.

DISCUSSION

Harry Tucker,* M. Am. Soc. C. E.—A careful study of this excellent paper prompts the following comments:

1.—It may be taken as axiomatic that a majority of the failures of pavements—particularly the rigid pavements—are due to unstable foundations.

2.—The unstable condition of the foundation is due largely to the volumetric change of the sub-grade materials.

3.—This volumetric change of the sub-grade material is due to its moisture content.

4.—The equivalent moisture content of a sub-grade material—a measure of the quantity of water which it will take up—depends approximately on its clay content.

Therefore, practically, the problem of obtaining a suitable foundation for a road may be solved by (*a*) providing a sub-grade material which has a low clay content, less than 25% if possible; or (*b*) reducing the moisture content of the sub-grade material. The latter requires some form of underground drainage to lower the ground-water level to a point where it will not be injurious either as free water or as water rising through capillary attraction.

Tile and blind drains have been long recommended by highway engineers to improve the condition of the sub-base, and the practice, for certain cases, is fairly well standardized. Their use is open to two serious objections: First, their universal use in all cases where improvement of the sub-base material is necessary would mean considerable increase in the cost of a road project; and, second, they are not always effective, for even where they are utilized serious damage may result from water which enters the sub-base from the shoulders and along the edges of the pavement, before it is carried off by the underground drains. Such unfavorable conditions would prevail during a wide range of temperature after a heavy rain, snow, or thaw. For these reasons the use of underground tile and blind drains is probably limited to cases where it is necessary to lower the level of free water, and where the question of cost is not so important.

The alternate method is that suggested by Mr. Rose, namely, the use of a sub-base material with a low clay content, less than 25% if possible. If this standard is adopted, it is probable that failures of pavement due to distortion of the sub-base will be infrequent. Is this method practicable?

The method outlined by the author would seem to be the most practicable for a great many of the Southern States. It will be found that the clay content of many of the top-soil roads is 25%, or less, indicating that this material would be ideal for a sub-base. This was pointed out by C. M. Upham, M. Am. Soc. C. E., some years ago in his description of the progressive system of building highways.† In this system all highways were graded to the standard for hard surface construction, but were surfaced with top soil or sand clay. When traffic needs demanded hard surface pavements, and finances permitted it, the top soil was used as the sub-base. In this manner a suitable sub-base material would be obtained without increasing the cost of the road to any great extent. This method requires that the ground-water level be kept at a reasonable depth below the pavement, so that it will not cause damage as "free water".

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* Prof. of Highway Eng., North Carolina State Coll., Raleigh, N. C.

† *Engineering News-Record*, Vol. 88, p. 202.

The use of top-soil material for the sub-base will help in decreasing pavement failures. Mr. Rose has indicated a number of other soils which may be used for the same purpose, and has pointed out how the survey maps of the U. S. Bureau of Soils, together with his tri-axial diagram (Fig. 11), will enable highway engineers to select a suitable material for the foundation of highways. It is thus a problem for the individual engineer to select a suitable sub-base, consistent with the type of pavement used, the traffic needs, and the cost. It is a fruitful field of research for every highway engineer.

WINSLOW H. HERSCHEL,* ESQ. (by letter).—When one has to deal with a mushy or pasty material like wet soil, to which neither the laws of hydrodynamics nor of elastic solids may be readily applied, it is fortunate when any progress whatever can be made. Mr. Rose is therefore to be congratulated that he has to some extent brought order out of chaos. On account of the complexity of the subject it is no surprise that any proposed method fails under certain circumstances. It may be of interest to consider some possible causes for these exceptions.

The author shows that the clay content is indicative of the field moisture equivalent; that this, in turn, shows the shrinkage value of the soil; and that it is the shrinkage value which is most important as a criterion for the value of soil to support pavements. He states that "* * * the displacement of pavements seemed to be caused principally by the distortion of the sub-grade in one direction—vertically." While this may be admitted, the horizontal movements of the soil should not be overlooked.

In railroad work the phenomenon of subsidence is recognized as important. Material dumped on the right of way may settle out of sight and eventually re-appear at a considerable distance to the side of an embankment, as an "island" in what would be considered solid ground. Evidently, this involves horizontal flow of the soil, and such horizontal flow takes place in the sub-grade soils of many highways, even though not to such an extent, nor in such a spectacular manner. The amount of flow depends on the consistency of the soil, and it is questionable whether any test mentioned by the author, including the more elaborate tests of compressibility, elasticity, and permeability, will adequately define the consistency of the soil. Mechanical analysis and the determination of the clay content are inadequate. Clays differ in many respects, including the percentage of colloidal matter which they contain and the state of flocculation of this colloidal matter, and these factors are not without influence on the consistency of the soil.

Consistency may be defined as the property which determines the distortion-force relation. In a simple viscous liquid the rate of distortion or flow is proportional to the force or pressure which causes the flow, and the viscosity may be expressed by a single numerical value. With a plastic material like soil, there is no such proportionality and at least two quantities are required to define the more complicated flow-pressure relation, that is, to define the consistency of a plastic material. Space does not permit a detailed explanation of the methods of measuring and expressing the consistency of plastic material,[*] but for the purposes of this discussion it will be assumed that consistency depends roughly on two quantities, namely, *A*, the pressure required to start the flow; and *B*, the rate of flow after motion has begun.

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* Associate Physicist, U. S. Bureau of Standards, Washington, D. C.

If it is admitted that consistency should be considered in tests of sub-grade soils, and that it is better to measure this composite property directly rather than to infer it from other tests, then it is a question whether information is required regarding Quantities *A* and *B*, or whether one alone will give adequate, even if not complete, information.

Perhaps in the case of asphaltic surfacing for highways, and surely in the case of soil under bridge piers, where an initial distortion may be as bad as complete failure, knowledge of Quantity *A* alone is sufficient. When, however, the soils under pavements are considered, from an economic standpoint it is impracticable to make all roads as solid as bridge piers. Rather is it desirable merely that the yielding of the sub-grade soil shall be small and gradual enough so that resurfacing will be necessary only on account of wear of the road surface, and not because the lateral flow of the soil, combined with its volumetric changes, takes away the support from the surfacing so that it goes to pieces under heavy traffic.

The rule to keep water out of the foundations is as good as ever, but again from economic reasons, perfection cannot be reached, and it is desirable to spend the available money where it is most needed and will do the most good. What appears to be necessary is the determination of the consistency of sub-grade soils with various percentages of moisture, that is, curves showing the variations of Quantities *A* and *B* with the percentage of moisture for each type of soil. Then the problem becomes one of finding at what percentage of moisture a dangerously "soft" consistency occurs, and in adopting such types of design as will maintain the moisture content below this value.

CHARLES TERZAGHI,[†] M. AM. SOC. C. E. (by letter).—In the history of every applied science it is possible to distinguish three successive stages. In the first stage, due to limited experience, the ultimate solution of the problems involved seems to be within easy reach and attainable within a few years. In the second, a broader knowledge of the empirical facts is acquired and the unsuspected complexity of the phenomena almost completely destroys one's confidence as to the possibility of discovering anything resembling simple relations or fairly valid laws. In the third stage, a separation is achieved between essential and unessential facts; and, instead of universally valid laws of Nature, modest rules are established with a small but well-known range of validity.

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[*] F. P. Hall, "Methods of Measuring the Plasticity of Clays," *Technologic Paper No. 234*, U. S. Bureau of Standards, 1923; W. N. Harrison, "Controlling the Consistency of Enamel Slips," *Technologic Paper No. 356*, U. S. Bureau of Standards, 1927; Winslow H. Herschel and Ronald Bulkley, "The Measurement of Consistency as Applied to Rubber-Benzene Solutions," *Proceedings*, Am. Soc. for Testing Materials, Vol. 26, Pt. 2, pp. 621–633, 1926; and Winslow H. Herschel, *Proceedings*, 6th Annual Meeting, Highway Research Board, pp. 413–416, 1927.

[†] Prof., Technische Hochschule, Vienna, Austria.

Mr. Rose's stimulating paper essentially deals with the first stage of the sub-grade science, in which he has played the part of a pioneer. At present, this science is passing through the worst section of the second stage, and it may be interesting to review briefly the extent to which views have changed during the seven years since 1921 when Mr. Rose made his valuable field studies in the Pacific Northwest. It usually requires outsiders to discover the shortcomings in new methods of reasoning, and the outsiders were, in this case, the Russian road engineers, backed by a long and brilliant tradition in soil science. In 1924, a few years after Mr. Rose made his investigations, the Russians started to organize their own sub-grade service (Highway Research Bureau of the Central Department of Local Transport of the People's Commissariat of Ways and Communications), and tried, as a matter of course, to avail themselves of everything that others had done in this field, particularly of the American methods. They recognized at once the weakest point of past practice. Obviously, it resided in over-estimating the importance of the properties of the soil as such. They knew from experience that the same soil may behave very differently, depending on the geological, hydrographical, climatic, and topographical conditions. They knew that no soil survey can serve its purpose unless it includes a general study of the locality, the data concerning the soil proper, such as mechanical analysis, and that soil constants represented merely a part of what is needed.

In general reconnaissance work, the Russian engineers proceed from the most essential to less essential items and, according to their technique, the quality of the raw material of the soil ranks among the less essential ones. In brief, they summarily rejected what Mr. Rose presents in his paper as Point 1 of his "Summary", protesting against what they called the conception of the soil to be "a chance association of pulverized matter". This attitude may be somewhat exaggerated; yet, it contains a great deal of truth. American engineers have learned that during the last few years. Data such as those presented in Table 2 certainly looked promising at the time when they were obtained; yet, the table is followed by the statement that "the exceptions to the rule [that the linear shrinkage of the soil be the critical test for sub-grade quality] have been noted in previous papers by the writer." Since the days when these "previous papers" were published, experience has been acquired quite rapidly. The number of exceptions was found to increase alarmingly, and—imperceptibly but irresistibly—the "center of gravity" of attention shifted from the rules toward the exceptions. There are obviously many more factors which influence the behavior of the road than one could possibly suspect seven years ago, and these are by no means as easily reached as is, for instance, the linear shrinkage. To give a few examples: The reports published during the first stage of the sub-grade science simply distinguished between good, doubtful, and poor sub-grades, and the readers were delighted to be offered such a clean-cut distinction. However, a more sophisticated generation started to inquire what the terms, "good, doubtful, and poor", really mean. The answer to this question was found to be rather perplexing because experience has shown that the same soil, from the road engineers' point of view, may be very good or very bad, depending on whether it is encountered in a cut or in a fill. In other localities, the order is reversed, inasmuch as the transportation of the soil from the cut into the fill decidedly improves its quality. Some of the worst frost troubles on New Hampshire roads were encountered on soils that should have been excellent considering that their linear shrinkage was less than 1.

Another factor, which was not yet considered in the earlier days, was the soil profile. Mr. Rose merely dealt with the older conceptions according to which the road engineer was satisfied to investigate the nature of the material serving immediately as a foundation; that is, the sub-grade. It is what one could call a two-dimensional conception of the problem. However, within the last few years, more and more attention has been paid to the fact that every natural superficial soil deposit consists of three superimposed layers, namely, the A, B, and C horizon (top-soil, subsoil, and parent material), with very different properties, and that the behavior of the road does not merely depend on the quality of the sub-grade proper, but also to a large extent on both the thickness and the properties of the layers beneath. As a consequence, the U. S. Bureau of Public Roads adopted, quite recently, the practice of combining the work of the survey party with the work of a soil scientist whose sole duty consists in exploring the soil profile and obtaining other data which are not disclosed by the laboratory test. This new policy was the inevitable outcome of the growing insight into the complex character of the sub-grade problem, and it is practically identical with what Russian engineers have done since 1925. The new policy requires the development of novel methods for expressing the results of field observations on road maps which show, not only the character of the narrow strip of land to be occupied by the road, but also the adjoining belt, together with its hydro-geological characteristics, and the stratification of the soil from the surface down to the parent material.

Similar imperceptible, but nevertheless quite revolutionary, changes have taken place within the last few years in the engineer's attitude toward the soil tests themselves—a change which is hinted at in the last part of the paper. When the first soil tests were completed, many years ago, each test represented a kind of patent medicine—its causative relations to the others being practically unknown. It was believed that a single test or two was sufficient to express, for practical purposes, the character of the sub-grade. The effects of a similar belief depreciated the value of the otherwise excellent work of the Swedish Geo-Technical Commission performed in 1914-22. Since then, however, engineers have learned that the situation is by no means so simple. Soils which, for instance, have the field moisture equivalent and the linear shrinkage in common may be very different in all their other aspects; and these other aspects may have on the quality of the sub-grade an equal or more important influence than the linear shrinkage. This behavior is particularly conspicuous with soils that represent a very poor sub-grade in spite of their low linear shrinkage, as, for instance, many of the soils in Northern New England. Hence, the efforts of investigators gradually turned from the original goal of correlating the results of one single soil test with the behavior of the roads to a broad investigation of the manifold properties of the soils—of their physical meaning, their inter-relation, and their bearing on the results of the various soil tests. Thus, not more than a few years ago, it was still believed that the Rose test expressed essentially the same properties as does the centrifuge moisture equivalent. A more careful analysis, backed by a knowledge of the underlying physical facts, disclosed that the two tests have very little in common, in a physical sense. Attempts were made to establish a mathematical relation between the silt content and the moisture equivalent. At present, engineers know the reason why there cannot possibly exist anything but a very crude statistical relation between these two quantities, valid merely for the average of a great number of different test results, but certainly not for individual materials.

The first results of these broader investigations* reflect, clearly, the utter complexity of the situation. They disclose the necessity of exploring the field even more thoroughly before one can dare to decide which and how many soil tests will be required for identifying soils reliably and at a minimum amount of time and labor.

These supplementary statements by no means curtail the merits of this paper. It undoubtedly has a lasting value as a lucid presentation of the early tentative efforts to solve the sub-grade problem, and of the opinions which served as guides. The test that bears Mr. Rose's name still ranks to-day among the simplified soil tests that deserve serious consideration.

D. P. Krynine,† M. Am. Soc. C. E. (by letter).—The writer's intention is to treat only three specific details in Mr. Rose's paper.

First.—He believes that the distortion of the soil is not only vertical, but also horizontal. Inasmuch as the greatest part of the horizontal forces are counterbalanced, the visible influence of a bad sub-grade is mostly, but not always, equivalent to the action of great negative vertical forces only, such as in the case of swelling.

Second.—The author states that field moisture equivalent is indicative of the shrinkage value, which in turn is important for determining the value of the soil. The content of clay by mechanical analysis, according to the author, may be considered equal to the field moisture equivalent. In other words, if the sand particles in a soil are separated from the clay, and the clay content is determined, there is a criterion for judgment of the quality of the soil as sub-grade. In reality, this is not so. The mechanical analysis and the application of soil maps in connection with Feret's tri-axial diagram seem to be insufficient for the purpose.

Third.—The author cites the example of a 2-in. Warrenite wearing surface on a 2-in. new stone course over a 6-in. old macadam road. He states that this surface "was not heavy enough to withstand the traffic under the existing adverse soil conditions and the inadequate surface drainage and sub-drainage". The old macadam had naturally worked in the same conditions of sub-grade and drainage. Therefore, there may be two basic cases: (*a*) before the construction of the Warrenite the old macadam had been in good condition, and hence the deterioration of the Warrenite is not due to the bad sub-grade; (*b*) before the construction of the Warrenite the old macadam had also suffered from the sub-grade distortion, in which case the question arises as to why the new Warrenite surface has been constructed over a base that was known to be in bad condition; naturally, there may be a third case (*c*) in which the traffic on the Warrenite has become greater than it had been on the macadam. If this is so, one may be led to the conclusion that a poor sub-grade can be tolerated unless the traffic attains a certain volume.

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* "Present Status of Subgrade Soil Testing," by Messrs. Hogentogler, Terzaghi, and Wintermeyer, *Public Roads*, Vol. 9, No. 1, March, 1928.

† Research Associate in Soil Mechanics, Dept. of Civ. Eng., Yale Univ., New Haven, Conn.

ALBERT C. ROSE,* ASSOC. M. AM. SOC. C. E. (by letter).—The writer desires to express his appreciation of the able discussion made by Mr. Tucker. It is gratifying to learn that Mr. Tucker's experience and that of another eminent road builder in one of the Southern States, confirm the findings of the sub-grade studies made in the Pacific Northwest.

The writer agrees with Mr. Herschel that the horizontal movement of the soil should not be overlooked. That Mr. Herschel's observation is correct is confirmed by the writer's experience in highway grading operations at the mouth of the Columbia River, in Clatsop County, Oregon, with respect particularly to tideland soil deposits. In common with the experience of other highway and railroad engineers, the writer observed that embankment slope stakes, placed at a predetermined distance before the fill was made, would be found, after the embankment was constructed, to have been thrust outward and upward for a considerable distance by the lateral and upward pressure at the toe of the slope caused by the weight of the superimposed material concentrated near the center of the fill. In many cases, this final distance between the slope stakes was found to have increased to a considerable degree beyond the measurements indicated in the original cross-sections.

The writer believes, however, that the need for the consistency test advocated by Mr. Herschel applies largely to the undrained tideland soils, in which there is a distinct flow of the wetted material, the area of which in the Coastal States is insignificant when the percentage is compared with the total area of the upland mineral soils. Because of this restricted area of the tideland soils, composed mainly of decomposed organic matter, it is believed that the consistency test is of secondary importance. In the drained tideland the lateral movement is eliminated to a considerable degree, and in the upland soils, by adequate drainage provisions, it should be possible to restrict the adverse action of the sub-grade soil principally to the vertical movement caused by the expansion or contraction brought about by alternate freezing and thawing or by variations in the moisture content; neither of which actions is considered to involve flow.

Professor Terzaghi calls attention to a statement made by the writer that, "the exceptions to the rule [that the linear shrinkage of the soil be the critical test for sub-grade quality] have been noted in previous papers by the writer". The insert in the brackets is the impression gained by Professor Terzaghi from the text. This impression, however, is erroneous, as the context of the original paper will show. What the writer stated was not that the exceptions to the rule that the linear shrinkage of the soil was the critical test for sub-grade quality, had been noted in previous papers (although this was true in a subordinate sense), but that the field moisture equivalent was taken as a simple and rapid indication of what the linear shrinkage values would be for identical samples of soil, and that exceptions to this statement had been noted in previous papers. This relation between the linear shrinkage value and the values of the field moisture equivalent have been discussed at considerable length by the writer in previous papers.*

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* Highway Engr., Bureau of Public Roads, U. S. Dept. of Agriculture, Washington, D. C.

The writer agrees, however, that there are exceptions to be found to the statement that the linear shrinkage value is the critical test for sub-grade quality. For example, Mr. E. R. Hoffman, formerly Construction Engineer for the Washington State Highway Department, reported that the maximum heave caused by frost under a concrete pavement in the irrigated section of Yakima County, Washington, was approximately 5 in. The sub-grade where the maximum heave occurred, was largely sand and gravel, a material in which the linear shrinkage would be negligible. The water-table, however, was in close proximity to the base of the pavement, and the frost penetration was approximately 9 in. at the edge, as compared with 15 in. at the center, of the pavement. The 5-in. heave referred to by Mr. Hoffman was specified to be the excess of the rise in the center over that at the edge of the pavement, as determined from the original crown of the surface. No spirit-levels were available, however, to determine the total heave.

Mr. Hoffman believed that the explanation of the heaving of the center of the pavement might be found in the excessive depth of the frost penetration at that location, and that this condition might be accounted for in part by the exposure of the center of the pavement when the snow blanket was removed so as to keep the road open to travel. The snow piled along the edges was considered to have been a protection against frost. Mr. Hoffman stated that there might have been other contributing causes, such as the condensation of the greatest amount of moisture under the center of the slab.

The unusual drainage conditions existing on this project, however, should preclude it from consideration as an exception to the general rule that clay soils (those with a relatively high linear shrinkage value) constitute unfavorable highway sub-grades. The testimony is overwhelming that the quality of a sub-grade may be determined largely by its clay content, as evidenced by the written experience of numerous highway engineers.

To recite the practice of actual field investigators, it is recognized that the most favorable period in which a sub-grade survey may be made is during or immediately following a heavy rain, when drainage conditions are at their worst. At this time it is generally a relatively simple matter to anticipate the location of the unfavorable sub-grade soils by their muddy condition as compared with the well-drained appearance of the more favorable sub-grades. To approximate this natural phenomenon, involving rain, the writer believes that the simplest and most practical test consists of the addition of water to a dried sample of the soil. The field moisture equivalent seems to be the nearest approach to this natural condition, since it was discovered that it represented the maximum moisture content of a well-drained soil even immediately after a heavy rain. This statement applies to a sample taken beneath the surface so as to eliminate the effect of any ponded surface water. Because of this natural relation the field moisture equivalent was adopted as the logical basis for adding comparable quantities of water to soils, with varying sizes of grains, as a preliminary to making the linear shrinkage test—which is believed to represent the fundamental adverse characteristic of a sub-grade soil.

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* "Practical Field Tests for Subgrade Soils," *Public Roads*, Vol. 5, No. 6, August, 1924; "Field Methods Used in Subgrade Surveys," *Public Roads*, Vol. 6, No. 5, July, 1925; and "The Present Status of Subgrade Studies," *Public Roads*, Vol. 6, No. 7, September, 1925.

The writer believes that the field moisture equivalent is the simplest, most rapid, most logical, and most practical test which has yet been devised to determine the general nature of a sub-grade soil. The close agreement of test results on identical samples by different investigators has been noted as another element in its favor, because of the small degree in which it involves the personal equation of the operator.

Professor Terzaghi is misinformed when he states that the soil profile was not considered in the writer's earlier investigations. On the contrary the *Bulletins* of the U. S. Bureau of Soils, upon which the writer based his method, for many years had included soil profiles as a part of the information with regard to the various soil types. Furthermore, the original investigations made by the writer on pavement failure were carried on in compliance with a standardized questionnaire form distributed to the Field Offices by the Headquarters' Office of the U. S. Bureau of Public Roads. The instructions on these mimeographed forms required that samples of the soil be forwarded to the Washington Office; that the horizontal and vertical location of the sample be given; that it be stated whether the sample represented the material extending at least 3 ft. below the road surface; and the depth at which a more pervious or impervious layer existed beneath the material representing the sample. This necessitated considerable drilling with a hand soil auger with pipe extensions for reaching depths in excess of 6 ft. The general practice, as shown by the records on file in the Bureau, was to bore down to the impervious layer. This involved drilling, in some cases, to a depth of 20 ft. A careful statement of the character of each soil layer was noted on a soil profile plotted for the length of the test section of road.

Later, when the method for making sub-grade surveys was developed by the writer, it was recognized that soil borings were essential.* As a general rule, however, since the top or immediate subsoil contained the greatest content of clay it was considered that the procedure could be simplified by considering these layers to represent the most adverse condition. Where actual observation, however, indicated that the lower subsoil, as shown, for example, on the side slopes of excavations, contained the largest percentage of clay, this material was used as a representative sample of the most unfavorable sub-grade soil. Where the cuts and fills did not exceed 6 ft. the soil profile could be found often by referring to the data contained in the *Bulletins* of the U. S. Bureau of Soils. The more recent *Bulletins* of this Bureau also give the soil profile down to the parent material.*

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* "Field Methods Used in Subgrade Surveys," *Public Roads*, Vol. No. 5, July, 1925.

An example of the boring and stratification sheets based on the data obtained in the earlier tests may be found in the report† of the sub-grade failure under a bituminous pavement on the Oregon Trail between La Grande and Hot Lake, Ore. This investigation was made in September, 1920. The borings showed the existence of a sand and gravel stratum from 4 to 8 ft. below the base of the pavement. The surface soil, up to 1½ ft. in depth, consisted of black loam—practically a gumbo—and beneath this there was a layer of white sand varying in thickness. The unfavorable subsoil, however, was found to be a quaking stratum between the white sand and the sand and gravel layers. This quaking layer was composed of greenish brown sandy clay grading into green coarser sand as the depth increased. The layer was evidently the remnant of an old bog or marsh. This conclusion was borne out by the geological formation of the Grande Ronde Valley in which the test was made. This valley is about 25 miles long by 20 miles wide, and is practically level, sloping slightly down hill from west to east at the rate of 10 ft. to the mile. The Blue Mountains enclose this triangular basin on all sides, towering to a height of 8 500 ft. above sea level. Geologists conclude that in prehistoric ages the mountains cracked, forming a tremendous fissure into which streams from the surrounding hills gradually deposited eroded material so as to fill the enormous cavity. The continual deposits through the ages finally formed the present broad level valley situated at an elevation of 2 700 ft. above sea level.

The pavement mentioned failed rapidly under the traffic of heavy trucks hauling material for the construction of an adjacent section of roadway. All the evidence indicated that the failure was caused by the vertical and lateral displacement (under concentrated loads) of the boggy, soft, and mushy intermediate layer of soil. The project illustrates conclusively the necessity of borings because, in this case, the adverse nature of the sub-grade was not apparent on the surface.

Professor Terzaghi's fine differentiation between the centrifuge moisture equivalent and the field moisture equivalent is interesting and instructive. The writer remains of the opinion, however, that for all practical purposes of testing sub-grade soil the identity of the two tests is sufficiently close so as not to affect materially conclusions based on the specified relation. This opinion is formed after considering the findings that have been made since the test was first promulgated. For example, in the three original papers in *Public Roads*, previously referred to, the writer showed that the test was applicable principally to soils with field moisture equivalent percentages varying from 20 to 30%, since these limits included the sub-grade soils of doubtful nature. For values less than 20% the soil was considered sufficiently coarse-grained so as to provide a good sub-grade, and if the percentage exceeded 30 in any quantity the soil was believed to be unfavorable as a sub-grade.

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* "Field Methods Used in Subgrade Surveys," by A. C. Rose, *Public Roads*, Vol. 6, No. 5, July, 1925, p. 94, and Footnote 4.

† Unpublished report on file in records of the U. S. Bureau of Standards, Washington, D. C.

This conclusion is confirmed first by the most recent researches of the U. S. Bureau of Public Roads. For example, a curve comparing the values determined on identical samples of a number of soils for the two moisture equivalent methods has been published.* This curve shows that the two tests agree remarkably well between values of 15 and 35%, although there is considerable divergence from the 45° relation above 35 per cent.

The second confirmation of the writer's original statement was rendered by Professor Terzaghi in explaining the physical significance of the various highway sub-grade soil tests. In this paper† the relation between the two tests was shown by means of a pressure-voids-ratio diagram. The difference between the two values, in the typical case cited, was represented as 0.003 (in terms of voids ratio), one ordinate indicating the value 0.625 and the other, 0.622. Since the higher value represented the standard centrifuge test, the field moisture equivalent was graphically indicated to be less than 0.5% below the standard centrifuge value. It would seem, therefore, that all the evidence to date tends to confirm the identity claimed by the writer for the two tests.

In rebutting these contentions made by Professor Terzaghi, the writer does not wish to leave the impression of failing to appreciate the inestimable service that he is performing in his chosen field—the mathematical interpretation of soil physics. On the contrary, the writer believes that Professor Terzaghi's papers, concerned as they have been largely with expositions as to the limitations and scope of the various sub-grade tests, have done much to clarify what had been hitherto a hazy conception of the subject on the part of many highway engineers. The writer also agrees with Professor Terzaghi that the soil tests, in themselves alone, can never be made to indicate the quality of the sub-grade, because of the impossibility of duplicating in the laboratory the conditions that exist in the field. The writer believes, therefore, that the center of attention, concerning the whole matter of highway sub-grades—which has been focused for the most part on laboratory tests in the past few years—should now be shifted to the scientific study of sub-grade soils in the field. It would be unfortunate, indeed, were more valuable time taken up with minor objectives, when the major objective is to identify the character of various sub-grade soils, so as to place in the hands of the field engineer methods through the practice of which there may be built road surfaces best adapted to local conditions. The sub-grade investigations will then perform their greatest usefulness in pointing to ways and means of reducing the ultimate cost of construction and maintenance.

The difference of opinion expressed by Professor Krynine in the second paragraph of his discussion is caused perhaps by a misconception of the statements made by the writer. The writer asseverates his opinion that most of the evidence published to date points to the conclusion that the adverse nature of a sub-grade soil in general increases with the clay content. That there are exceptions to this statement it is admitted. As Professor Krynine does not state the ground for his difference of opinion, the writer assumes that he takes exception to the statement from the standpoint that local conditions such as drainage, climate, and traffic, largely determine the manner in which a sub-grade soil will re-act regardless of its clay content. With this conception the writer heartily agrees; he never wished to convey any other impression. On the other hand, perhaps Professor Krynine's difference of opinion may be caused by a lack of similarity in the methods used on the two sides of the Pacific Ocean in determining the clay content.

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* "Present Status of Subgrade Soil Testing," by C. A. Hogentogler, Charles Terzaghi, and A. M. Wintermeyer, *Public Roads*, Vol. 9, No. 1, March, 1928.

† "Simplified Soil Tests for Subgrades and Their Physical Significance," by Charles Terzaghi, *Public Roads*, Vol. 7, No. 8, October, 1926.

With respect to Professor Krynine's third paragraph, his Assumption (*c*) is believed to represent the actual facts in the test under consideration. At the time this section of the Pacific Highway in Washington was built (in 1911), it was impossible for the authorities in control of road construction to foresee the tremendous increase in traffic which would occur within the brief space of a few years. In 1911, American roads in the Western States, as well as in other sections of the country, consisted of disconnected sections built for the most part adjacent to the larger cities and towns. The adoption of a comprehensive system of State highways in Washington, in common with many other States, was still in the argumentative stage, and it was not until 1921 that the Federal Aid System was enacted into law by the Congress of the United States.

In conclusion, the writer wishes to express the need of simplifying the sub-grade problem and the necessity for placing the subject on a plane which will make the results useful to the field engineer. He believes that the plan on which his original studies were made, has been and still is of distinct value to the field engineer as a working hypothesis on which to base the selection of the kind and design of the road surface for a given location. The outline of the operations made in carrying on these sub-grade surveys is recited here for a definite purpose, as follows:*

Report Form.—

1.—Brief description of highway.
2.—Information with regard to adjacent roads:
    (*a*) Location.
    (*b*) Type.
    (*c*) Sub-base and character of soil.
    (*d*) Year built.
    (*e*) Present condition.
    (*f*) Summary.
3.—Description of this project: Type, etc.
4.—Topography of adjacent terrain.
5.—Geological structure of adjacent terrain.
6.—Soil types encountered:
    (*a*) Data on borings.
    (*b*) Results of soil analysis.

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* "Field Methods Used in Subgrade Surveys," by A. C. Rose, *Public Roads*, Vol. 6, No. 5, July, 1925.

Report Form.—(Continued):

7.—Drainage conditions.
8.—Climatological data.
9.—Traffic census.
10.—Recommendations:
    (*a*) Drainage: Surface and sub-surface.
    (*b*) Pavement design and kind of surfacing.
11.—Remarks.
12.—Photographs.

The writer believes that this outline indicates that his original investigations were not made with the idea that the soil tests alone would determine the efficiency of a sub-grade soil. On the contrary, he always recognized—and so stated—that the climate, traffic, drainage, and other local conditions should all be considered in the final selection of the surfacing. The method outlined for making a sub-grade survey was intended to be submitted as a working hypothesis that had been found successful in the Pacific Northwest. It is gratifying to learn that the findings of others have confirmed the results and that the study of sub-grade soils is rapidly assuming a growing significance as an integral part of the new science of highway engineering. In order that this new science may have a solid foundation, however, the writer believes that there is a need for the accumulation and analysis of facts, with regard to typical sub-grade conditions, on a country-wide scale.

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Paper No. 1724

GENERAL CONTRACT SYSTEM VERSUS SEGREGATED CONTRACTS*

By Ward P. Christie,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Edward W. Bush, William T. Lyle, B. F. Jakobsen, C. C. Muhs, W. J. Barney, Lazarus White, L. S. Stiles, Gustave R. Tuska, T. Kennard Thomson, H. M. Dougherty, W. A. Starrett, Robert Ridgway, R. E. Bakenhus, Wilson T. Ballard, and Ward P. Christie.

Introduction

By laboring constantly for greater economy of design, the Civil Engineering Profession has developed an amazing fund of technical knowledge. This knowledge, applied, has made possible the structural growth of the country. Well-deserved commendation has come to the profession. However, it is possible that the very intensity of thought exerted along technical lines has paved the way for criticism, as well as for commendation.

The thought has been expressed more or less frequently, that the Civil Engineering Profession in general is too thoroughly engaged in its technical pursuits to school itself in other necessary learning. The inference is, that the engineer usually does not become acquainted with those elementary principles of business, which any man should know, whose life work is concerned with construction. It is not presumed, of course, that the professional man should array himself with a business education, but rather that he should have a better understanding of the fundamentals of business management and the competitive forces with which he deals.

It has been stated, whether properly so or not, that many economies gained through increased technical knowledge are lost by unsound practice followed in the actual administration of construction. Some grounds for such a statement seem to have been evidenced by the many conferences of construction. For example, certain customs used in awarding and administering contracts are evidently sound and economical, while others are inherently wasteful. It is now recognized that not only the amount of a bid, but even the actual cost of construction may be influenced greatly by general provisions and payment clauses of a contract. In fact, many factors affecting the ultimate cost of a project transcend entirely the field of design.

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* Presented at the meeting of March 7, 1928.

† Engr., The Associated General Contractors of America, Washington, D. C.

One of these factors, which is the subject of this paper, seems particularly important. It is the relationship that should exist between the various responsibilities of construction and the control of operations. In private corporate enterprise, this relationship is well established, but in the administrative work of construction, the issues are still debated. An effort is here made to throw some light on the subject by analyzing the two methods of contracting in current use. For want of a better name they are designated as the General and the Segregated Contract Systems.

Each of these two systems has its proponents and opponents among the business agencies of construction, and the engineer or architect should be on his guard to discern the motives of the individual or business group. Much has been written concerning them, and considerable has been enacted into codes of practice or law, but through it all apparently runs a forgetfulness of those who pay the bill.

The general contractor capable of financing and managing a project in its entirety prefers the general contract, because it creates a demand for his specific service. A sub-contractor may prefer the segregated contract because it enables him to deal directly with an owner, thus escaping certain abuses which can be practiced by an unscrupulous general contractor. Some architects or engineers may believe that their conceptions are carried out more satisfactorily by segregation, whereas others unquestionably lean toward it to escape the backfire of inadequate plans and specifications.

It is apparent that either the general contract or the segregated contract can be utilized to the specific convenience of one or another of the various agencies of construction, but as all those agencies are presumed to serve the owner, either public or private, and as the owner's money makes construction operations possible, it seems logical that his interests and not those of any individual, profession, or business group, should be the criterion for judgment. Using that conception as a premise, therefore, the following analysis of the basic features of the two systems is offered, with the hope that the conclusions drawn may at least stimulate collective thought.

Some of the points discussed pertain more directly to building work than to general engineering construction, but from them the engineer can readily draw proper analogies for his own field of operation. The subject is subdivided into the following sections:

Definition of a General Contract,
Definition of Segregated Contracts,
Function of the General Contractor,
Financial Responsibility,
Legal Considerations,
Delay in Completion,
Relative Cost to Owners,
Relative Quality, and,
Accident Prevention.

Definition of a General Contract

Building Construction.—The general contract in building operations is an agreement entered into by an owner and a single construction agency whereby that agency agrees to deliver to the owner a structure fully completed in accordance with the plans and specifications of the designer. It places upon this single agency, known as a general contractor, the task of supplying by such agencies as he may select, all construction equipment, materials, labor, special services, and appliances necessary to complete the project; and, furthermore, obligates this agency to protect the owner against loss from suits, claims, liens, and other encumbrances.

The outstanding characteristic of the general contract in comparison with other types is centralized responsibility for financing, for general management, and for detailed timing of installations whereby the cost of the work and the time of completion may be controlled according to some rational and comprehensive plan.

Engineering Construction.—The general contract in engineering construction establishes duties and responsibilities similar to those in building work, but may cover either an entire project, or merely one of a number of similar sections into which the work is divided by reason of its magnitude. The significant feature of the general contract in engineering construction, as elsewhere, is that it places under a centralized management and co-ordinating control those operations which depend upon each other or interlock in time and place.

Obviously, some engineering projects have natural segregations of work entirely independent of each other, with respect to interlocking, which makes them suitable for separate contracts. Such projects, of course, are excluded from this analysis, as they do not conflict with the principles herein stated with respect to interlocking work.

Definition of Segregated Contracts

The term, segregated contracts, is applied to the system of contracts whereby an owner enters directly into agreement with a number of independent contractors for the construction of the component parts of a project. Under this system no contractual relation exists between the independent contractors and, generally speaking, each is an agent of the owner. The function of co-ordinating their operations, or, in other words, the general management of the project, is performed by an agent of the owner.

In comparatively few cases of building work is the complete segregation of contracts carried out, as it is generally recognized that there must be some kind of central management by an agency that is financially responsible. Certain parts are awarded to concerns that usually operate as sub-contractors, and the remaining major portions are awarded to some principal contractor. This partial segregation, however, is subject to the same reasoning as the complete process, because the difference is only a matter of degree.

Function of the General Contractor

When properly functioning, the general contractor finances the work during construction, performs certain parts of the work himself, and co-ordinates the operations of all agencies engaged on the project, so that it may be constructed properly as a whole. Also, he assumes certain risks, peculiar to a business based on selling futures, and certain others, also, which arise merely from the presence of a number of different contractors operating on the same site. Assumption of these risks in reality constitutes an insurance business of the most hazardous kind, for which no commercial company would attempt the underwriting. As an owner cannot obtain commercial insurance against them, they are pooled and assumed in the service of the general contractor. Among the contingencies against which he protects the owner are: (*a*) Increased cost of construction; (*b*) delay in completion; (*c*) injury to workmen and the public; (*d*) patent infringement; (*e*) injury to adjacent property; (*f*) damage from the elements; (*g*) imperfect material and workmanship; (*h*) liens and the abuse of lien laws; (*i*) default on subordinate contracts; (*j*) unknown underground conditions; (*k*) demands for extortionate wages; (*l*) fire and theft; and (*m*) labor, material, and transportation shortage.

Contrary to popular conception, the principal function of the general contractor is not to erect steel, brick, or concrete, but to provide skillful centralized management for co-ordinating all the various trades, timing their installations, and synchronizing their work according to some predetermined plan. In other words, the essence of general contracting lies, not in the broking of sub-contracts or the furnishing of labor, materials, and equipment, but in the highly specialized function of management, the success of which depends on the personal skill and direction of capable executives.

Due to the great number of different trades entering into construction, particularly building work, all of which must be co-ordinated with the main skeleton and principal elements, those principal portions are usually performed by the general contractor's own forces. To carry out his commitment with certainty, he must have actual physical control of these parts. Mere legal or contractual control without physical facilities for performing almost any part of the work does not assure completion within the time or at the cost intended.

The mere broker of contracts cannot successfully perform the service of a genuine general contractor. If one branch of the work falls down, he cannot perform it himself, but must seek a new agency to perform it, which, at best, means serious delay. The same is true of engineers and architects, even if they have the experience and ability to manage a project. In some instances, engineers or architects have successfully performed this management function, although they have done so, not by reason of their technical skill and training, but in spite of it. They were good construction executives, as well as technicians, with an understanding of business management—a combination rarely found in either business or the professions. Ordinarily, the professional engineer's or architect's management of construction may reasonably be expected to succeed about as often as the contractor's execution of engineering or architectural design.

FINANCIAL RESPONSIBILITY

The general contract holds one single agency, a general contractor, financially responsible for the completion of a project within a given time and at a specified amount. If either cost or time should over-run, it is the contractor's and not the owner's loss. This is not the case under segregated contracts wherein the designer performs the general management of the work. He does not presume to guarantee the cost or pay the penalty for delay, and, of course, should not be expected to do so, as his services are supposed to be professional. When an engineer or architect awards interlocking portions of a project to several contractors and himself performs the managerial duties necessary to co-ordinate their work, he is not exercising professional duties, but the duties of a business executive.

Perhaps the designer himself has not always perceived or understood the contingencies and liabilities which a segregated job places on an owner; nevertheless they are there, economically, ethically, and legally, and the owner cannot escape them. If the designer assumed these liabilities, the case would be different. He would then be acting as a broker, whose profit is justified, not only by the service performed, but by the risk of business which he has assumed. In that event he might well repaint his shingle to signify that he is a "Contracting Engineer", or a "Construction Contract Broker", for he is no longer strictly speaking a professional man.

The liabilities here mentioned are not theoretical or imaginary, and, in many cases, have ultimately cost the owner enormous sums. Independent contractors operating each according to his own convenience cannot avoid interference with one another. The work of each depends to a great extent upon the performance of another, and friction between the different superintendents is almost inevitable. Issues arise over the use of hoists, the laying out of conduits, storage of materials, the installation of safety devices, and other matters. Such conditions result in confusion on the job, mutual interference among the independent contractors, injection of extra expense, and the filing of perfectly legitimate claims against each other or the owner. The owner generally cannot escape payment of such claims against him, because his own representatives are named as being responsible, and because there is no financially responsible agency between himself and the independent contractors to absorb the loss.

From the owner's viewpoint, however, the broker is a "safer bet" than the professional engineer or architect; for he guarantees the price, usually has business experience, and, in the last analysis, must pay the cost of his own mistakes. When the designer splits up the work among a group of independent contractors he is acting as a broker with all a broker's financial safeguards and guaranties removed. This aspect of segregated contracts, although apparently not recognized by some professional men who have favored them on occasion, is a most vital consideration with respect to the interest of a construction owner.

If the professional man desires to engage in business, he has of course a perfect right to do so, and doubtless should do so if his capabilities lie in in that direction; but to exercise the business function involved in segregated contracts, and to let an owner proceed in such cases without informing him of his liabilities, does not have the earmarks of fair play.

Legal Considerations

To a certain extent each contractor holding an independent contract with an owner is his agent. Consequently, if one interferes with another's operations or injures him, the owner may find that he must stand suit jointly with the offending contractor. Even if the offender's agreement provides that he shall protect the owner against loss from actions brought by other contractors, such an agreement may not actually insure the protection. Obligations and responsibilities devolving upon each of the several contractors on a specific project, are difficult to determine, and the offending contractor can usually manage to shift the responsibility for injury to the owner's own representative. Moreover, the true obligations and equities of the various agencies of construction are so confused in the minds of American Courts, that there is little hope of pinning liability where it belongs. This condition is evidenced in part by the steady trend of construction toward the settlement of disputes by arbitrators familiar with the industry.

In addition to damage suits, there is also the question of criminal liability in event of accident or disaster. Rarely, if ever, has a Criminal Court in the United States convicted a contractor, designer, or owner, whose criminal negligence or wilfully dishonest actions have resulted in death to third parties. Under segregated contracts, a lack of clearly defined responsibility between the designing and construction agencies, and failure to pin these responsibilities where they belong, have made it nearly impossible to convict a guilty party. The injurious effect of such a condition upon the integrity of the entire industry must be obvious.

If an owner is sued when segregated contracts are used, he is very likely to be doubtful whether he should stand suit with the offending contractor, or join with the plaintiff against him. A properly drawn general contract, however, centralizes all responsibility in a single agency whose obligations are relatively clear. That agency can be reached at law and held responsible by an owner should the performance be upset by ignorance, negligence, or dishonesty of any agency on the work. In other words, the mere splitting up of interlocking work among a group of independent contractors, under the supervision of the owner's own representative, may place him in an extremely unsafe legal position.

Delay in Completion

Completion of a construction project within the time set is nearly always an important consideration to the owner, and, in many cases, is a vital one. Owners are not particularly interested in collecting penalties, but in obtaining their projects when expected. If the various operations are not "time-scheduled" in advance and constantly synchronized on the job, delay is inevitable.

All other factors being equal, it seems almost axiomatic that responsibility and control when vested in a qualified general contractor, whose own money is at stake, will insure timely completion more certainly than when vested in the owner's professional adviser. Without an existing organization trained for the work, and without equipment or other facilities for actually taking over some backward portion, the latter is practically helpless to control the time of completion.

The points here mentioned are generally recognized by engineers and architects, but there is another point that is quite commonly overlooked, namely, delays which constantly menace construction operations in and around cities by reason of difficulties with labor. The complications of union and non-union labor, as well as open and closed shop conditions, may readily paralyze the separate contract job. No one cause is more fruitful of delay and increased cost than this, particularly when contractors from different localities are engaged on the same project. Instances are not uncommon of a project having been delayed for weeks, because one of the independent contractors on an open shop job is obliged to employ only union labor. This condition needs little comment.

Ramifications of labor problems are not confined to single companies, single communities, or single crafts. A strike on one job may be called for causes arising on some job in a distant part of the country. The labor problems of construction, like mechanical installations, require special skill and experience. They require proper contact between the employers in the various trades, functioning of their trade associations, conferences with labor representatives, sometimes an organized resistance, and, in fact, a continuous working together of all elements aside from their contact on the work. The handling of labor has become probably the most difficult phase of the construction business in many localities, and the professional man is not in a position to handle the situation.

Relative Cost to Owners

Various proponents of segregated contracts have commonly sought their adoption by citing specific cases wherein the total of separate proposals on a construction project was less than the proposal for a general contract. This, they claim, demonstrates the economy of the system, but, at the same time, they fail to state that there are far more cases wherein the general contract bid is lowest. Also, there is another angle to proposal prices, which overshadows initial comparison, as follows: Assuming for the sake of analysis that plans and specifications are correct and complete, then, under a general contract, the contract price is all that the owner is required to pay; but when several independent contractors are engaged on interlocking work under the direction of an architect or an engineer, the sum of their prices rarely, if ever, constitutes the total cost to the owner. The extra expenses, as already suggested, arise from delays, interference, misplaced work, and lack of co-operation between the independent contractors. Under a general contract this over-run, if present, is absorbed by the general contractor. Under segregated contracts, it can fall only upon the owner.

In addition to the concealed expense of segregated jobs, there is also to be considered their effect upon competition. In receiving proposals for a general contract the owner obtains a double competition: First, the competition of sub-contractors dealing with general contractors; and, second, competition between general contractors. Frequently, it occurs that the general contractor, considering a proposal for some part of the work too high, substitutes his own figures, and brings the cost within them. It is commonly recognized that he usually receives lower bids from the various trades than an owner dealing directly. He has his regular contacts with sub-contractors who wish to deal with him in the future, whereas the owners may never employ them again.

A great many of the most experienced general contractors do not bid on projects likely to be segregated, because they feel that efficient performance of their work in such event is practically impossible. The award of a single integral part to some non-co-operative individual, disrupts and increases the cost of their work. Thus, the owner loses valuable competition which would often bring in more satisfactory proposals.

Some members of the mechanical trades in seeking to establish segregated contracts have represented that general contractors add a margin to the sub-bids, for which no service is given, and that this margin—called profit—can be saved by dealing directly with sub-contractors. Some owners may not detect the fallacy of the statement, but the analytical mind of a professional man should expose it. Entirely aside from the management service of the general contractor, there are many other expenses on the work which this so-called profit must defray.

Inquiry among representative construction companies shows that the usual margin added to sub-contracts is about 5 per cent. This amount, although in some cases it may contain an element of profit, is in reality a service charge, to compensate for such items as superintendent, night watchman, hoisting, storage space, protection from weather, temporary heating, safety work, and other overhead expenses which are not included in the sub-contractor's bid. When it is recalled that the architect in private practice charges about 3 or 4% for handling sub-contracts direct, yet provides none of these things except superintendence, it is obvious that the 5% contains no great amount of profit.

Under partial segregation, the principal contractor knows that he will be required to furnish these items and, therefore, includes them in his estimate. Then, when the designer adds to this his 3 or 4% for supervising separate work, the owner pays almost a double charge for general supervision. All the independent contractors, if they have had previous experience with segregated contracts, are very likely to include some allowance for the contingencies of a poorly co-ordinated job. Income tax reports show that the average net profits in contracting have been less than 2%, and that approximately 30% of the companies made no profit in 1924; therefore, there is evidently some room for doubt about the saving of this profit under separate contracts.

The subject of cost under the two systems may be analyzed and debated endlessly, because opportunity is never afforded for a true comparison on identical projects under identical conditions. If, however, experience in other lines of business counts for anything, and if the common axiomatic principles of business management and economics are founded on fact, segregated contracts cannot avoid certain inherent inefficiencies and expenses which are eliminated by the general contract system.

Relative Quality

Workmanship and materials under the general contract system receive a double inspection; one by the engineer or architect to protect the owner and another by the general contractor to protect himself—for he is responsible to the owner for the work of sub-contractors. The supervising agency evidently values this latter inspection, as it is required by a great majority of contracts.

Under separate contracts, the engineer or architect can doubtless secure a satisfactory quality in the work of a specific contractor, but quality in the specific trade does not insure quality in the project as a whole. Unless the various parts have been properly timed, joined, and finished as a whole, the value of the project is impaired.

A well-known architect has stated, with respect to buildings, that there never had been, and probably never would be, prepared a complete and perfect set of plans. Some adjustment, alteration, and modification in the field is necessary. Doubtless, the same is true of engineering plans. This condition seems to necessitate some central agency in the contract to produce a structure that is integrally sound.

Proper timing of operations is a necessity on which the quality of a building project particularly depends. Conduits must be placed, not when the plumber or the electrician feels like doing so, but when forms and structural parts are ready to receive them. The heating plant is essential to the drying of walls and the execution of certain other work. Plaster, marble, tile, and painting are also dependent on the heating plant and, if it is not installed at the proper time, the results are not satisfactory.

Under complete segregation, proper joining and finish, or cutting and patching as it is commonly called, is extremely difficult to produce. Each trade seems to consider itself at liberty to bore holes, but feels no responsibility for their filling and finish. Under partial segregation, where some principal contractor is held responsible for such work, this is also a problem, as such contractor lacks control over those who cut the structure. A number of construction companies that have been consulted declared that they would like to be rid of responsibility for the mechanical trades, were it not for the fact that, when those trades operate under independent contracts, efficient operation and satisfactory conduct of the work as a whole is impossible.

Accident Prevention

Owners may not know it, but they are the ones who pay the costs of accidents in construction. The cost of injuries and deaths finally finds its way back to them through high production costs, increased premium rates, and general inefficiency in construction. Safety work is, therefore, not only a humanitarian obligation, but an economic necessity. It can only be carried on successfully when centrally supervised and enforced throughout a project.

When several independent contractors are operating on the same site, intelligent, co-ordinated safety measures are neglected, and the rate of accidents increases. Responsible contractors have stated that proper safety measures are practically impossible without some centralized management on the job, whereby some one answerable for his mistakes can, if necessary, force the installation of safety measures.

In some States, failure to comply with existing safety regulations precludes protection under the Workmen's Compensation Act. The contractor and the owner in case of accident may be jointly sued for twice or triple the limitations of the Act. If the contractor sued is responsible and is acting in the capacity of general contractor, the owner is safe. No doubt exists concerning who is responsible for all parts of the work; but if there are a number of independent contractors, responsibility may not be located or placed, and the owner may be obliged to pay the damage.

CONCLUSION

As previously mentioned, individuals in one or another group participating in construction prefer one or the other system of contracting. The reasons of some are unselfish, whereas those of others are distinctly selfish. No group is entirely one way or the other, and, in each, there is a tendency to blame the deficiencies of human beings on a system of contracting.

When the sub-contractor seeks to establish segregated contracts because an unprincipled general contractor tries to drive his bid down to that of an irresponsible competitor, he should realize that dealing directly with an owner is not a remedy. Owners or their representatives commonly do the same to general contractors, or for trifling causes hold up large sums of his money. They will do likewise to the sub-contractor. A number of representative sub-contractors have stated that they prefer the general contract, when the owner employs a responsible general contractor.

The professional man who represents that an owner can profit by eliminating a general contractor on the grounds that he performs no necessary service, is at best uninformed. Construction service is composed of certain elements which represent relatively certain expenses. They may be re-arranged among various agencies, but they cannot be eliminated. The significant point to be considered is, which of these agencies can perform the various services most efficiently and most satisfactorily to an owner. No one presumes that a man trained to the contracting business can step into design and render proper service. Neither can the designer step into general contracting, without actual contracting experience, and perform proper management service for an owner.

The general contract system was not suddenly conceived and launched on the populace, but grew to meet the requirements of owners, through a long period of years. It has endured in the hands of individuals who were unorganized, when all industry around them had developed powerful trade associations. By its endurance has been demonstrated the soundness and economy of centralized construction management, and it should not be cast aside merely to suit the convenience of a specific trade or profession.

Rather than attempt to correct any group or individual annoyances by scrapping a system which is both theoretically sound and practical, the responsible elements of each group should co-operate in correcting abuses, particularly as those abuses will prevail under any system, until all elements concerned with construction wage war on ignorance and unethical practice within their ranks.

DISCUSSION

EDWARD W. BUSH,* M. AM. SOC. C. E. (by letter).—The author very admirably presents a subject which can well be studied at this time by engineers, architects, and others in charge of the letting of contracts, because, during recent years, there have been organized campaigns in certain localities to require contracts for public buildings to be let by the segregated method. Many reputable sub-contractors doing plumbing, heating, roofing, electrical work, etc., have claimed that they frequently do not get fair prices for their parts of the work because, after the general contractor is awarded the job, he makes the sub-contractors come down to the figures presented to him by, perhaps, a group of irresponsibles. Such practice is deplorable and against the best interests of the construction industry considered from the point of view of the owner, the engineer, or the architect, or the contractors interested. On the other hand, trying to cure this evil by letting segregated contracts is substituting a worse practice for one not as bad.

Other methods can and should be used to correct these bad practices. Even where a general contractor does pinch the sub-contractors a little, the owner still has the general contractor's contractual obligations standing between him and any loss or damage caused by a sub-contractor not doing his part. The right kind of a general contractor will select the right kind of sub-contractors and very often they have worked together for years and each knows just how he fits into the other's organization. This makes for economy, which is reflected in the price paid by the owner as well as the profits obtained by each contractor. No engineer or architect can fit a sub-contractor into a general contractor's organization as well as the latter, and the work will suffer, as explained so well by the author, unless there is a centralized authority and responsibility. The owner has the best chance to obtain the maximum value for the money expended if he gives a building contract to one bidder after competitive bidding. This is the time-tested method under which so much work has been and will be performed.

It is known by the writer that most of the high-grade general contractors doing building work possess the requisite knowledge to estimate closely the cost of the portions of the work which will be done by sub-contractors and such cost estimates are actually prepared as a check on the sub-contractor's bids. Unless the engineers and architects have an estimating ability equal to that of the general contractors, the owner's interests are not conserved by segregated contracts.

Referring to the 5% mentioned by the author as being added by the general contractor to the bid of the sub-contractor, it is of interest to compare this to the 5% fee very generally allowed receivers and administrators of estates on all monies coming in and going out, and these receivers and administrators are also allowed expenses and the employment of expert services if such are needed. It requires a high degree of technical ability and much tact to keep the sub-contractors of a large building job up to the mark on progress and quality of work, and if the services rendered by the general contractor were compensated for in proportion to their true value, as compared with the value often allowed by law for the services rendered by receivers or administrators, the contractor could add, not 5% to the sub-contractor's bids, but 20 to 50 per cent. The small fee actually charged by the general contractor is gross and from it must come the overhead expenses, so his fee really nets him a very small amount of profit, if any. For this fee the general contractor must assume full responsibility on the work performed by the sub-contractors. The owner must necessarily pay some one to exercise the authority needed to correlate the efforts of several organizations, and it is to his advantage to place this authority squarely on the general contractor rather than attempt by segregated contracts to save a little and make the engineer or architect carry the authority without indemnification to the owner should anything go wrong.

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Even if the general contractor does add 5% to the bids of sub-contractors when making up his bid, he may not add very much more than this to the cost estimate of the part of the work to be done with his own payroll. It is the net profit he will take from the entire job which interests the owner rather than what precise percentage is added to the estimated cost of the various parts. The author has explained that building construction work is generally taken on a small margin of net profit. The writer, after reviewing a large number of contractor's financial statements, confirms this opinion.

A few years ago the contractors bidding on a large school were instructed to file separate bids for all or some of the parts, such as the main structure, the plumbing, the heating, the plastering, etc. They were told that the entire work would be let to one general contractor at an amount determined by adding the lowest bids received on the component parts; and if the general contractor did not choose to award sub-contracts to those who put in the lowest figures, these figures, nevertheless, governed the price the general contractor would receive for these parts of the work. It is difficult to imagine a more vicious form of a letting as it would place full responsibility on the general contractor and, at the same time, take from him the right to select the sub-contractors whose work he guaranteed.

WILLIAM T. LYLE,* M. AM. SOC. C. E. (by letter).—The writer believes it to be unwise for an engineer to take the attitude of proponent or opponent of either contract system. Each has its advantages and each, its disadvantages. The author, who favors general contracts, states that "some engineering projects have natural segregations of work entirely independent of each other, with respect to interlocking, which makes them suitable for separate contracts". With this unquestionable statement, he dismisses segregated contracts from his analysis and proceeds to establish the merits of the general system.

The author establishes a good case for general contracts in building construction, basing his argument on the interlocking character of the work; but the question will arise as to when and to what extent the elements of a construction job are interlocking. In analyzing this question three considerations arise: First, the parts of the work may be so far removed in space and relationship as to be suitable for segregated contracts, as the author thinks proper; second, they may be so closely related as to constitute an organic whole for which the general contract only is suitable; and, third, they may lie between these extremes, either toward the former or toward the latter. The engineer in charge must make the decision.

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It is natural for an engineer to favor the general type of contract. The writer has usually preferred it for the simple reason that responsibility for the timing of the work, legal responsibility, accident prevention, etc., is removed to a considerable degree from his shoulders and placed on those of the general contractor. In company with many of his fellow practitioners, he may have sometimes erred in this preference. The personal convenience of the engineer may not be for the best interests of the owner.

In discussing this question the professional and business equipment of the engineer must be considered. There was a time when, as a mere technician, he was poorly equipped to act as executive, but that time is passing. The modern engineer is taking to himself functions of the promoter and executive and thereby fitting himself for many kinds of important work for which he was formerly poorly qualified. He is becoming more and more a man of affairs. This change in the make-up of the practical engineer is reflected in the professional schools wherever increasing emphasis is placed on the preparation of a humanistic, economic, and administrative character.

The writer does not favor the categorical "no" of this paper. There are many large jobs, and smaller ones too, interlocking to a certain degree, and yet so spaced or spread out as to be adaptable to the segregated contract. The engineer in charge of such contracts will have more responsibility but, at the same time, can render greater service. By letting separate contracts, small contractors will have a chance to deal with the owner direct and a double profit may be eliminated. At any rate the work performed will probably be of a better quality. It is often true that the small contractor prefers to deal with the owner direct rather than with a general contractor of whose business character he is suspicious.

The writer is not undertaking to state any "yes" or "no" in the matter, but merely to emphasize the importance of weighing the pros and cons, in cost, security, and promptness of execution. He believes that both systems are useful.

B. F. JAKOBSEN,* M. AM. SOC. C. E. (by letter).—The author somewhat forcibly emphasizes the advantages of the general contract system, as a system. The writer's experience leads him to believe that the important element is the ability and experience of the particular general contractor as against the particular construction engineer. The general contractor assumes certain risks and responsibilities, but the owner pays for these in any case, and it is part of the construction engineer's duties to see that these things are properly arranged.

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The author admits that in some cases engineers have performed management functions successfully, not by reason of their technical skill and training, but in spite of it. It is difficult to understand this statement, and it is not clear just what the author intends to include in his expression, "technical skill and training". Of two men, one of whom has technical training and the other has not, but who are otherwise equal, there can be no doubt about which one is the more desirable. Some of the most successful contractors the writer knows are technical graduates, and this training does not seem to have affected adversely their ability to understand the management business.

In fact, it would seem self-evident that the executive who understands the reasons for the design, would be *prima facie* better qualified to carry it out. It is quite true that it is pitiful to see a theoretically competent man dabble with practical problems with which he has no experience; but it is still more pitiful to see the practical man dabble with self-conceived theories and insist that designs must be changed to accord with these strange notions of his. The latter case is of much greater frequency than the former, and when the purely practical man has gotten some theoretical conception in his head, based upon his many years of experience, he is generally not amenable to reason. On the other hand the manager of construction who has a fair understanding of the theories involved in the design, as well as of the factors influencing construction, is often in a position to assist the designer materially and thus affect a saving without other adverse effects.

The danger of over-emphasizing the advantage of the general contractor is that undue weight is likely to be given to the financial details at the expense of realities. Good accounting is valuable, but the accountant must always understand that his work is after all of a secondary nature. Many, if not all, of the larger engineering corporations which claim to be management experts are intrinsically financial experts rather than construction managers. Judging from such work as the writer has seen, neither the quality nor the cost of their work has been such as would lead any one to give their system any decided preference.

When the Constitution of the United States was being formulated, Madison insisted that this was to be a Government of laws, not of men. That may be true in a narrow sense and by comparison, but after all it is men who decide what the laws mean in each particular case, and that is what counts; or, as Pope said,

"For forms of Government let fools contest,
Whate'er is best administer'd is best."

It is quite the same in the construction field; at best a system can make up for very little compared to the ability and skill of the individual in charge. The owner needs to look for the best man rather than for the best system.

C. C. Muhs,* M. Am. Soc. C. E. (by letter).—Contrary to the author's opinion, the number of contractors with technical training is legion. His statements that:

> "* * * many factors affecting the ultimate cost of a project transcend entirely the field of design." * * * "In some instances, engineers or architects have successfully performed this management function * * * not by reason of their technical skill and training, but in spite of it." * * * "When an engineer or architect awards interlocking portions of a project to several contractors * * * , he is not exercising professional duties, but the duties of a business executive."

show a bias that is, fortunately, rarely held by a contractor with technical training and somewhat infrequently by the "practical" contractor.

In the early history of construction work, plans, in the sense of drawings, were few and sketchy, and the architect (master builder) or engineer (one who devises and controls) was practically the general foreman on the job and controlled in detail the processes used and the manner of their execution. During that epoch there was little segregation of the crafts, and even later when a contract was "letten" to John Doe, carpenter, to build a certain bridge, it was not assumed, therefore, that he knew more about building bridges than the man who designed them, but merely that by this device less detail supervision would be required by the engineer. At present, in Continental Europe, Asia, South America—in fact, practically all over the globe except North America—all engineers and architects are contractors, in the sense used by Mr. Christie, and all contractors are either engineers or architects. In view of this fact, Mr. Christie's statement that "he [the designer] might well repaint his shingle to signify that he is a * * * 'Construction Contract Broker', for he is no longer strictly speaking a professional man", loses its significance.

The problem then "boils down" to a matter of expediency, because engineers are not inclined to cavil about contractors having encroached upon their field of management.

As to things affecting expediency, which Mr. Christie denominates the owner's interest, the author believes that some architects or engineers "unquestionably lean toward it [segregation] to escape the backfire of inadequate plans and specifications." The writer believes that those guilty of such omissions lean to the general contract as more likely to cover up their sins. Both practices, of course, are based on a fallacy, because the owner pays either directly or indirectly.

It is admitted that "mere legal or contractual control without physical facilities for performing almost any part of the work does not assure completion within the time or at the cost intended", but since no general contractor in building work performs more than a few of the many details involved he is not in much better case in this respect than the engineer or the architect.

The writer takes issue with the statement ending, "* * * because a broker does not customarily offer effective guaranties".

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* (Muhs & Co.), Chicago, Ill.

Further, the author states, "* * * to let an owner proceed in such cases without informing him of his liabilities, does not have the earmarks of fair play". If such practice exists, this statement would be a serious indictment. A considerable experience has failed to show that it is at all frequent.

There is a fallacy in the statement that, "the true obligations and equities of the various agencies of construction are so confused in the minds of American Courts that there is little hope of pinning liability where it belongs".

The owner's liability to sub-contractors and the public is only slightly affected by the existence of a general contract, his security in that case being the reliability of the general contractor—not the contractor's existence; and there are also reliable sub-contractors.

Furthermore, the writer questions the logic of the following statements: (*a*) "If the various operations are not 'time-scheduled' in advance and constantly synchronized on the job, delay is inevitable"; (*b*) "Without an existing organization trained for the work, * * * the latter [the adviser] is practically helpless to control the time of completion"; (*c*) "The complications of * * * labor * * * may readily paralyze the separate contract job"; (*d*) "* * * cutting and patching * * * is extremely difficult to produce"; and, (*e*) "* * * safety measures are practically impossible without some centralized management * * *."

These are the functions that an engineer performs as an agent of the owner, and, with the exception of patching, he can perform them as well, inherently, as the contractor.

To state that "the professional man is not in a position to handle the situation", is merely to express an opinion.

As to the general contract case, Mr. Christie states:

> "* * * The essence of general contracting lies * * * in the highly specialized function of management, the success of which depends on the personal skill and direction of capable executives."

This is wholly true, but since most of the superintendents, estimators, and contract men who have become executives in this sense are employees of technical talent, and, since such talent may be just as readily employed by the practicing engineer, it makes, not a case for general contracting as a whole, but for each individual concern.

The following statements appear in the paper:

> "Assuming for the sake of analysis that plans and specifications are correct and complete, then, under a general contract, the contract price is all that the owner is required to pay; * * * over-run, if present, is absorbed by the general contractor."

This is only partly true, because the greater part of such excess is extra work or "unforeseen delays", which will be charged to the owner in any case.

Certainly, "he [the contractor] has his regular contacts with sub-contractors", but so has the engineer.

It is probably true that "this amount [5%] * * * is in reality a service charge". However, the inference that it includes such items as superintendent, watchman, protection, temporary heat, etc., is wrong, as all engineers, architects, and contractors know. Hence, of this 5% a greater part is profit than would at first appear.

If the fact that income tax reports indicate only a 2% average profit for contractors means anything exceptional, it means that contractors are not handling their work well. This, the writer doubts. The same conservative source of information forces the gloomy conclusion that "nobody never made no money nohow".

Many of Mr. Christie's observations are true, but many others contain grossly exaggerated and unfair inferences, as when he states: "Conduits must be placed, not when the plumber or the electrician feels like doing so, but when forms and structural parts are ready to receive them". The inference that, under the control of an engineer or architect, the project would run "hogwild" is so often repeated throughout the paper that it should not go unchallenged. The plaint amounts to this, that engineers, by a system of segregating contracts, are usurping the functions and encroaching upon the field of the general contractor. This is measurably true, but it is not entirely true. Under such conditions engineers would be usurping the functions of the general contractor as defined fifty years ago which he, in turn, usurped from the engineer a hundred years ago. There is ample field for the general contractor without his embarking on a campaign of belittling the ability and integrity of engineers to whom he owes at least as much as they owe him.

W. J. BARNEY,* M. AM. SOC. C. E.—Although connected with a building organization, the speaker would like to discuss this subject from the standpoint of an engineer. There is no question that, after reading this paper, engineers and contractors will realize that there is little economic, and little administrative, justification—taking the broad picture as a whole—for attempting to break up a general contract into a number of separate contracts. On the other hand, this problem has been discussed because it exists, and since it exists there must be some causes.

The speaker believes that the practice of segregating and letting separate contracts on building operations has arisen largely through a weakness in the Engineering Profession. This does not apply to a great many engineers; but there are a number in the profession to-day, particularly in the building industry, who are selling their services, not as engineers, not as technical advisers, not as men who can lay out a plant for a better operation, but purely and simply as a type of super-purchasing agent.

The speaker has known cases where engineers have said:

> "You give me the commission to build this building—not because I can lay out a plan so that you will have the best building for your money; not because I can design a more beautiful building; but because I can buy this building cheaper for you than you can buy it for yourself, or because I can buy it cheaper than my fellow engineers; I am the super-purchasing engineer."

A man selling on that basis grasps at straws to help him bolster up his argument. The factory owner says: "How can you buy this building cheaper?" And the engineer replies:

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* Pres., Barney-Ahlers Constr. Corporation, New York, N. Y.

"I can buy the building cheaper because I am familiar with sub-contracts. I can save the general contractor's profits; you can eliminate the general contractor. For my fee I will furnish you the plans and specifications. I will do all the buying and I will let the contracts separately and there will be none of this large, wasteful general contractor's profits in this enterprise."

Often the factory executive accepts this type of argument; and, consequently, the Engineering Profession to-day is losing its stability and professional character because of the introduction of this element of purchasing agent.

As to this question of the segregation of contracts, it will work itself out in the long run. If it has an economic justification, the speaker will cease to exist as a general contractor and will become a sub-contractor on the concrete skeleton. However, there is something foreboding to the Engineering Profession in this tendency, which, as a member of the Society, and as one who has been an engineer, the speaker dreads to consider. The profession should stop selling itself as a purchasing agent and go back to selling itself as a technical adviser.

Look at the subject from the standpoint of facts. Suppose that an engineer has sold the owner on the basis that he can do the work cheaper. What are the facts in the case? He sends out and gets bids on the electrical work, on the plumbing work, and on the elevator work; and he says to the general contractor: "I have no objection to your doing the same thing, and we will see if you can buy as cheaply as I can."

On a preliminary estimate no sub-contractor to-day practically ever gives his lowest price, because he has been trained in the hard school of experience, and he knows that when the contract is closed and the buying begins he is going to be asked to reduce his price somewhat, perhaps to justify the existence of a purchasing agent, and perhaps also to justify the idea that the engineer is a clever fellow who can always take a few dollars off. Therefore, the engineer is asking for a price and the contractor is also asking for a price. The chances are that the estimates will be the same, although the following possibility enters: The sub-contractor may sense that he is not going to deal finally with a general contractor, but with an engineer purchasing agent, and in order to put himself in a better position, he will give a price closer to his final proposition than he gives to the general contractor; but this seldom happens.

With these separate estimates—which the engineer totals as against the general contractor's bid—the general contractor guarantees his estimate and the engineer does not. The engineer then calls in the plumber and the heating man and the others, whose estimates may have amounted to $25 000, and says to them: "You can have this job for $20 000." Perhaps they haggle awhile and it is finally closed for $23 000. Then the purchasing engineer goes to the owner and says: "Here are the original bids that I received from the general contractor, and here is what I purchased it for; I have saved you 5 or 10%, and I have more than justified my fee." He has not done anything more than the general condition permitted both to the owner and the general contractor; but he has set up a false picture of conditions, and has created a myth that he can buy more cheaply.

There is no need to pursue the aftermath of this method of handling a building operation—its costly lack of co-ordination—because Mr. Christie has covered this phase fully. The speaker has no intention of suggesting that the engineer is not essential; he is very necessary. Nothing is more unpleasant for a general contractor than to be left to face the whims and misunderstandings of an owner without a competent engineer as an arbitrator; and the speaker always prefers to have a contract under the supervision of an engineer or an architect, provided he is competent and provided he has put his services on a professional basis and not on that of a purchasing agent.

Lazarus White,* M. Am. Soc. C. E.—The speaker has had approximately thirty years' experience, about half of which has been as an engineer administering general contracts—mostly public works—and the remainder as a contractor—partly as a sub-contractor and partly as a general contractor; so perhaps his experience has been more rounded, or has covered a larger field than others.

As a sub-contractor, naturally the speaker might want to deal directly with the owner, so as to escape some of the hard general contractors; but in the interests of the owner himself and disregarding the special interests the speaker believes that the owner is better off with the general contractor. He certainly would be with the average class of general contractor.

The speaker's firm has often taken work directly with the owner, using the owner's architect (who is really his representative) or an engineer purchasing agent. They are very punctilious, and after making a plan, they are so proud of it that they will not depart from it. The general contractor has not prepared the engineer's or the architect's plans, and he is, therefore, much more free to depart from them. The result is that the owner does not get what he ought to get; he does not get the experience and advice of the different specialists that are on his building.

In one very glaring case, an architect prepared a plan for a contract which was taken directly from the owner. This plan passed the Building Department; it provided foundations for the larger columns, but not for the smaller ones loaded to 40 or 50 tons each. The owner would not put those in. He said the plan had passed the Building Department and must be all right, and this was an extra and he was not going to pay for any extras. What should be done in a case like that? A general contractor would have "ironed" this out very quickly. There would be a new foundation provided for the loads and an "extra" that would be in the interest of the owner.

Of course, this whole subject comes down to the beginning of the matter; that is, has the contractor an economic justification? If the contractor cannot justify himself economically, he deserves no special consideration; but after long experience the general contractor has survived. The reason is this, that by and large, the general contractor's profits, although in special cases they may be large, will average a rather low percentage.

The efficiency of the general contractor—a man of peculiar type, who has usually fought his way up and is one surviving of a great many—generally is much higher than that of the owner, and is greater than the difference indicated by his fee. Suppose that he makes 5% on the general contract (3½% is more nearly correct). The owner, although he may be a very good clothing manufacturer, is not a very good builder. The fact that he is very good at something else probably means that he has not the ability of a general contractor, although he often thinks he has. He cannot make up that difference; he cannot make up the contractor's profit.

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* Pres., Spencer, White & Prentis, Inc., New York, N. Y.

It is the same way with the engineer, who, of course, will naturally think that he can make up the difference of a contractor's profit because he designed the structure, and he has investigated conditions and thinks he knows all about it. Perhaps he does; but he lacks certain abilities. If the engineer designs the job himself, he may want to save the contractor's profits.

This also applies to public works, where the community, through the engineer, endeavors to do the work directly. If they can do it by that means more cheaply than through a general contractor, they ought to do it. The speaker will say that although it may destroy his business, but he does not believe they can do it more cheaply. He has seen too much of public work and the way the public business is conducted.

Necessarily, when he starts a job, the contractor has more freedom. He selects his men; he does whatever is necessary to get the particular men he wants. He builds up an organization, and that organization has to make good and knows it, and usually does. Public work is not managed on that basis. It has to be done with a public organization, plus a few additions, and the same efficiency cannot be obtained that is usually obtained by a general contractor.

In connection with public works the speaker would say that the general contractors on subway works are really general contractors. It would be difficult to segregate the different contracts. The excavation, steel work, etc., could not be segregated underground. The only alternative to a general contract would be for the public to do its own work, which has the objections that have been stated.

L. S. Stiles,* M. Am. Soc. C. E.—Some time ago, the Company with which the speaker is associated, was having a large structure erected under a general contract. The general contractor, naturally, brought in several miscellaneous trades. Had it not been for the experience of the men handling this contract, in co-ordinating the trades so that all lines worked together without overlapping or lagging behind, it is the speaker's opinion that progress with the structure would not have been as satisfactory as it was.

The speaker's work is not confined to building projects, but also includes equipping plants with all kinds of machinery and apparatus and other operations of a miscellaneous nature. Very often, under these conditions, it is found that the general contractor fits the situation. The work can be started later. The beginning can be postponed until such time as it is felt that the adequate facilities for proceeding with the work are there, and if it is then to be in the hands of a general contractor—a good co-ordinator—progress is made.

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* Constr. Engr., The Brooklyn Union Gas Co., Brooklyn, N. Y.

The speaker knows of many cases of owners of different kinds of business who have been very successful in their particular line. Some of them have attempted to do the requisite building themselves when expansion was necessary. They thought that the general contractor would be somewhat in their way and that he would be collecting money for work that they could do themselves.

However, he has never known an instance where the owner did not get into "hot water" before he was finished with the work. The owner has his sphere; the engineer has his sphere; and so has the general contractor; and they should not clash at all. It is true the owner should have a proper engineer, but what is really important is that none of the parties should overstep its own bounds. Companies that are enlarging their plants and facilities by extensions may engage, now and then, in the building business, but the general contractor is always in the business and, because of this, he has better relations with the trades. The average sub-contractor, in working on an extension or a new project, is apt to feel that, while he secured this order, he may not get another from the same owner. However, if he is working for a general contractor, he feels that he will have to make good, because when that particular work is finished, the contractor will have other projects on which he would like to be considered.

The speaker thinks that the attitude of the sub-contractor is different when he is doing business through the general contractor and when he is doing business through the owner, and that this is one of the great advantages of having a general contractor. He has never known a set of plans and specifications to agree, in every detail, with the finished job. The specifications never exactly include all the features that the owner hoped eventually to have built. Because of this, there is bound to be some over-reaching and some under-reaching, and the general contractor is far better fitted to "iron out" these difficulties than an owner himself would be.

The cost to install a certain item under a general contract is always more than the credit which would be allowed if it were eliminated. The speaker has had many particular instances of this. Not long ago, on a contract, it was decided not to buy a certain quantity of a certain item. The contractor was quite agreeable to the cancellation, but he still insisted that he should receive his profit on the cancelled item. This does not seem a reasonable demand.

The speaker has handled millions of dollars of contract work. It has been let out, sometimes by segregated contracts and sometimes through a general contractor, depending on the particular conditions, and both methods have proved satisfactory. The important thing is to be sure that both parties comprehend the project and that the work is organized in the proper manner. When, in the judgment of the owner's engineer, the scope of the proposed work warrants it, the speaker believes that a high-grade general contractor should be allowed to handle the job.

GUSTAVE R. TUSKA,[*] M. AM. SOC. C. E.—The question has been raised, as to whether the client is to rely on the general contractor or on the engineer for success in the work undertaken. The speaker has, at various times, acted as general contractor and, at other times, as engineer. The differences in the qualifications necessary for a successful engineer and a successful general contractor are fully appreciated. Nevertheless, it must be remembered that many successful contractors were previously successful engineers and received an important part of their training as such. There is considerable doubt as to whether they would have been equally successful contractors without such previous training.

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[*] Cons. Engr., New York, N. Y.

In discussing this subject, it should be emphasized that all building construction does not belong to one class of work. The importance and magnitude of metropolitan building construction is fully appreciated, but there is a large amount of construction work concerned with special industries. This includes the various lines of factory construction where special machinery and special types of construction are required. In such cases the special experience needed to handle the work successfully is more likely to be found in the experienced engineer than in the general contractor.

From the discussion and from the cases cited by the author, one would be likely to reach the conclusion that most engineers handling construction work for their clients are exceptionally incompetent and that, on the other hand, most general contractors doing similar work are invariably absolutely reliable. Experience will show that there are efficient and inefficient engineers and capable and incapable general contractors and that, in the main, the success obtained will depend to a great extent on the kind of each class that has been employed on the work rather than on the class itself.

It may be of interest to contractors and engineers to know that both contractors and engineers have been described as experts using their best judgment, with the difference that the contractor backs his judgment with his own money, whereas the engineer backs his judgment with his client's money.

T. Kennard Thomson,* M. Am. Soc. C. E.—This entire discussion reminds one of the discussions thirty-five to forty years ago when the statement was frequently heard that "engineers could not make good contractors". This was before so many engineers had become very successful contractors. At that time the speaker answered such a statement by saying that he could sit down and typewrite a letter, taking a long time to do it; but if he did so for months, he would be as quick as an ordinary stenographer; and, by the same principle, a good engineer was not a good contractor until he had learned contracting by hard practical experience. If he then found that he had good business as well as engineering ability there was no reason why he should not excel as a contractor.

This has been abundantly proved by the experience of many engineers. The main reason why general contractors are worth while is that they have learned their business, have their organization, and, in addition, are financially responsible for what they do.

On the other hand, if there is no general contractor, the person who does the work of a contractor is merely an agent of the owner, who could not be successfully sued by his client, and is often untrained for the job.

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* Cons. Engr., New York, N. Y.

H. M. Dougherty,* M. Am. Soc. C. E.—The author has covered part of a very large subject, but his paper seems to have been limited largely to the construction of buildings (which the speaker has always looked upon as the province and problem of architects), and to construction in the Metropolitan District. Possibly, this is only natural in New York; but it must be remembered that the broad field of engineering and the allied business of contracting extends throughout the United States and Canada; and that the American engineer, with New York as his base, does work in South America, Europe, Asia, and Africa. The speaker is possibly qualified to comment by twenty-five years' experience which, starting as a Constructing Quartermaster in the Army, leads successively through Engineer in Charge for the Contractor on Government contracts with the Bureau of Yards and Docks, and Engineer in Charge and Resident Engineer for the Contractor on public utility works, to Construction Engineer and Chief Engineer in charge of large construction works for operating corporations—this throughout the United States, in Hawaii, and in South America.

The speaker believes that the viewpoint of many contractors may be expressed as follows:

> "We have grown into large contracting firms, and the Engineer, as an individual, is setting himself up as competent to do what we, as a corporation, advertise ourselves able to do; and what he, on account of professional ethics, may not advertise himself able to do. We feel that the Engineer as an individual must stay off the ground which we desire for ourselves. We as General Contractors will as, and if, necessary hire the Engineer; but we must be left alone in the field and the Engineer must be satisfied to be our employee; or the adviser and buffer between us and the owner."

In the case where the owner is not organized, or he cannot enlarge to take in construction; where he has not a purchasing department that he may bring to bear on the purchasing involved in construction; and where the life and size of the work are not such as to warrant him in purchasing or hiring the necessary equipment; the general contractor has not only a proper place, but a place which he fills with profit to himself and to the owner.

Where, however, the owner is an operating corporation which has a purchasing department capable of enlargement to take care of construction problems; where there is an organization which may be enlarged; and where there is work in hand which is either so continuous or of such size that there is warrant for investing in or renting proper equipment; a general contractor is out of place. For him to enter such situations complicates them and adds to the expense of the owner. The general contractor in these cases cannot use the knowledge of the operating force to the same full advantage as the individual engineer either in design or in construction; and the end is dissatisfaction both to the owner and to the contractor on account of the friction engendered.

That an engineer of wide experience may, with profit to these owners, be placed in charge of an engineering and construction department, taking the place of the general contractor, is to the speaker's mind not open to doubt or question. To lay down as a principle that the engineer of wide experience must move to the rear to make room for the general contractor, under any conditions, takes very debatable ground.

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* New York, N. Y.

The experienced engineer of to-day is not simply a man of science, not a man skilled in one particular line. He is an organizer; a man competent to carry on the entire business of design and construction; to originate methods; to carry them out; and to protect his principal in the matter of contracts, purchases, and labor contacts.

In the field described as not suitable for the general contractor, the writer holds a distinct brief for the individual engineer; and that his ideas are not entirely wrong is attested by the fact that so many public utility corporations, mining corporations, motor corporations, steel corporations, etc., find it to their advantage to have a department in charge of a competent and experienced engineer, to carry on the work the general contractor carries on in the contractor's proper field, for the owner who requires and needs the contractor's facilities.

W. A. STARRETT,* M. AM. SOC. C. E.—This paper addresses itself to the whole broad field of contracting, and comment, herewith given, refers only to that highly specialized branch generally known as large, metropolitan office buildings. Where such structures are under consideration, it has been the speaker's experience that the divided contract is most detrimental, and the unified, all-embracing contract is the only practical solution for the type of work in question.

It is true that certain architects, particularly those in the West, carry on, with considerable success, the function of the general contractor; but in doing so, they are, in effect, general contractors and are so organized. The risks to the owner may or may not be pointed out to him, but the responsibility must be obvious, and he evidently calculates accordingly.

Contrary to popular belief, contracting includes very little engineering work. If it were an engineering problem at all, it would be called business engineering, but the only point of contact between the two is that a knowledge of engineering is of value to a contractor but is in no wise essential, and it is likewise measurably true that wide experience in engineering in no way assists in considering the practical problems with which the contractor is confronted.

Successful contracting is based on wide experience and it is a most complex procedure in the field of tall, metropolitan buildings. As Mr. Christie has stated, the activity is almost wholly a managerial and co-ordinating function, but it also demands a going organization that is accustomed to "teaming" together, that understands the policies and precedents involved in the decisions and activities of the principals of the contracting company. The custom of the trade has a bearing. Contracting has grown up around sub-contracting, and it is beside the point to aver that it were better if this were not so. The whole trend of the industry is that the sub-contractor looks to the contractor for his contracts, his payment, his protection against injustice, and, most particularly, the skill of management which will enable him to get in, complete his work, and get out with a profit, not jeopardized by the risks of interference. From this situation has grown a relationship in which the best sub-contractors depend very measurably on the fair treatment and experience of the principal contractor. With one principal contractor a given sub-contract may be regarded as a hazard and risk, whereas exactly the same specified work may be considered without hazard if the general contract is handled by another and more favorably considered general contractor.

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In spite of all the care that is given to predetermination and pre-planning, modern buildings are changeable and changing, even while under construction. The contracting system that gives this change the greatest elasticity without exposing the owner to disadvantageous claims is the best one. It is to be borne in mind that the desire for change almost universally arises out of the needs of the owner, seldom out of the needs of the sub-contractor. An owner represented by an architect on a highly subdivided contract is at a disadvantage when this situation arises. On the other hand, a sub-contractor depending on the general contractor with whom he may be doing business year in and year out, considers the question of changes and substitutions more equitably. Sub-contractors naturally look with suspicion on owners who have never built before and will possibly never build again. They fear that if they are thrown into direct contact with the owners, the latter may prove to be either capricious or arbitrary in their dealings. Generally, owners have no standing with the sub-contractors in their dealings and properly may be regarded with apprehension as an enterprise is undertaken.

Whatever is stated herein refers especially to private contracting. It is recognized that the Government civil agencies may not have the elasticity to deal with building problems that a private owner has. It may be expedient, apart from the mere practical question involved, for a Government to adopt any of a number of systems.

However, there seems no real justification for the separation of contracts where large, complicated structures are concerned and where the small fee paid to a capable general contractor for his services is so sure to be returned many-fold by the economies of service and administration that he contributes to the operation.

ROBERT RIDGWAY,* PAST-PRESIDENT, AM. SOC. C. E.—The speaker is an engineer and not a general contractor. As Chief Engineer of the Board of Transportation of the City of New York, he has to do with the construction of rapid transit subways. The subway construction contracts may be regarded as examples of general contracts. The large construction contracts include all the items of work that make up the complete structure, such as excavation, concrete, water-proofing, and steel. They are let to the lowest bidder who can qualify, and he is made responsible for all the work to be done under the contract. He is the general contractor. The construction contracts do not include track, station finish, or equipment, which call for work of special character, which work is done under separate contracts.

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The subway work is generally divided into units of from ½ to ¾ mile in length, and of a value of from $4 000 000 to $8 000 000. It is believed by the Board of Transportation that contracts of this size are the most advantageous because they are large enough to attract responsible concerns and not too large to restrict competition. There are exceptions to this rule where the conditions require it. For example, the under-river tunnels are let in units that sometimes cost from $10 000 000 to $15 000 000, or more. The largest single contract let to date is the tunnel under the East River at Fulton Street. Including its land connections, the bid price is approximately $22 000 000.

It does not seem practicable to segregate these contracts; that is, to make separate contracts for the excavation, concrete, steel, and other items. If that were done the engineer would have to be the general superintendent in lieu of a general contractor, and there would be all sorts of interference between contractors, with resulting confusion, delay, and claims. Since they are public contracts, the Board of Transportation cannot follow the practice, common in private work, of choosing a number of qualified contractors and limiting the bidding to them. The work is open to general competition which means that the best qualified contractor is not always the low bidder, although the Board has been very fortunate in getting contractors of the character that it has. The Board has laid down the rule that the successful bidder must show liquid assets available for the job to the amount of 10% of his bid. This is to insure that the work will be carried on without the delay that always results from a lack of sufficient capital. The rule is a good one, and has the effect of eliminating those who are not financially able to carry the burden of the work. On only a few occasions, have contracts been awarded to other than the low bidder, and then only because the low bidder did not qualify.

A public contract is a rigid instrument and after it is entered into cannot be modified without an enormous amount of effort. Having this in mind the unit price form has been adopted for the subway contracts because that form provides flexibility and is believed to be fairer to both sides. A subway contract is a complicated one at the best. Not only must it take care of the building of the structure, but also of the underpinning of adjacent buildings and the restoration or reconstruction of all sub-surface utilities that exist under the busy streets of the city.

In other words, the street must be completely rebuilt, and provision must be made for paying the contractor for all this complicated work. Those who have had experience in this class of work know that it is almost impossible to predict what will be found under the surface of city streets. The ratio between quantities of earth and rock may be different from what is anticipated, and the quantities of the other items may be increased or diminished. This means that the contractor should balance his bid so as to provide for such contingencies. It is believed that the lump-sum form of contract would not be as fair because of the uncertainties.

Perhaps the speaker has strayed rather far away from the subject of this excellent paper, but he wants to point out that on the Board of Transportation work the construction contracts are general contracts. If the segregated type of contract were used the engineer would have to act as the general superintendent or the general contractor, and, generally, he is not trained along those lines. It seems better to continue with the system now in use and have the trained contractor do the work instead of giving it to the engineers to do by day labor or by segregated contracts.

There are times, however, when one wonders whether a trial of another method would not be warranted as, for example, when a technical contractor goes to Court and wins a large sum of money, not on equity, but because a comma is misplaced in the contract. That money might have been saved if the work had been done by day's labor under the direction of the engineer. On the whole, however, the contract system is believed to be better.

R. E. Bakenhus,* M. Am. Soc. C. E.—The speaker's experience has been entirely in the State and Government service and, since 1901, with the Corps of Civil Engineers of the Navy, largely in connection with the construction work of the Bureau of Yards and Docks. It has covered all phases of contract work from the standpoint of the owner and the engineer; the owner in this case being the Federal Government. It has also been the speaker's good fortune to have had much experience in connection with the construction of public works by direct employment of labor and by the method of individual contract, thus placing the officer in charge in the position of the general contractor.

It has been, therefore, very interesting to note the author's views in regard to general contracts. There is no question that the building business in general could not get along without the general contractor; he is a very necessary part of it.

With any given project to be undertaken it seems necessary to make an individual analysis of the situation. There are a number of cases in which a general contract does not best meet the situation; perhaps they are only a few, a small minority of cases. One or two examples, of opposite extremes in regard to size, occur to the speaker. One, of course, is the Panama Canal, in which case the letting of the work by general contract was very thoroughly considered and finally rejected, and it was done entirely under the supervision of engineers, who originally had experience, of course, with day labor. The other example was the case of a small power plant which needed rehabilitation and enlargement. The appropriation was so small that the work could not have been accomplished had all the items been lumped under a general contract. It became necessary therefore to make a number of contracts which dovetailed into one another and included in individual contracts such items as the following: (1) Foundations for boilers; (2) erection of boilers; (3) boiler settings; (4) purchase of electric generators; (5) purchase and erection of engines as a separate contract, but including the forcing of the generators on to the engine shaft; (6) purchase of pumps in open market; (7) erection of certain piping and equipment; and, (8) construction of new wing to boilerhouse.

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* Capt., C. E. C., U. S. N.; Public Works Officer, Navy Yard, Brooklyn, N. Y.

It meant much work and anxiety on the part of the engineer to handle the work in this way, but it was done successfully in this instance at a large saving in cost and without so much as a question about a single item throughout the progress of the combined work. The total cost of the Panama Canal was about $400 000 000, and that of the power plant just mentioned about $40 000.

Another instance in which the general contract did not seem to be advisable was the construction of the U. S. Naval Armor Plant at Charleston, W. Va. The decision to proceed with the construction of the plant was reached by the Secretary of the Navy in August, 1918, during the latter part of the World War and at a time when the naval building program was in progress. Orders were issued to prepare the designs which were completed during the next few months, and then further orders were given to go ahead with the construction work. In the meantime the Armistice had been declared. At that time the markets for labor and material were in such an uncertain state that the Department did not feel that the general contractor would be able to give a fair lump-sum bid. He might either over-estimate or under-estimate the conditions as to labor or material or both. A "cost plus" contract did not seem wise because it was a rather unpopular form in the Government service at the time. Therefore, the only decision left, as recommended by the speaker, was for the Bureau of Yards and Docks itself to become the general contractor.

Such factors as the steel frame work, steel sash, roofing, and a few minor items were made the subject of individual contracts. All the other parts of the work, including excavation, concrete foundations for buildings, furnaces and machinery, brick work, railways, roads, sewers, water piping, and electric installations were undertaken by the direct employment of labor. As the Navy Department had no organization at Charleston for such work, it became necessary to purchase equipment, establish a labor office, and purchase material as well as secure transfer of materials from other points where it was no longer needed by the Government service.

The work undertaken cost more than $10 000 000 and was completed successfully, on time, within the limits of funds available. It is hardly likely that such a result could have been accomplished at that time by any other method of construction. So far as known, it is the largest single enterprise in the line of public works construction, which the Navy Department has ever carried through as one operation.

It is very necessary to emphasize that for any project it is desirable to make a thorough study of the situation as to how it shall be carried through and then decide what is the best means to accomplish it. In such a situation there must be taken into account not only the project itself, but also the personnel that would be available to handle it; the market condition as to labor and material; and what general contractors are available and able to handle the project under consideration. Above all, it is necessary to take into account the experience of the engineers or other personnel who would be placed in charge of the work, either under the contractor or under direct execution by the owner. In the three cases which have been mentioned by the speaker, it has been shown that there were particular reasons why the general contract was not the method adopted.

There is no question that in the majority of cases the general contractor has the experience and ability to carry through complicated or simple construction work better than it can be done by the owner or engineer. The various reasons for this have been so clearly brought out in the paper that it is not believed necessary to repeat them.

Some of the larger corporations, as well as the Government service, have construction problems which are of a continuous character, such as railway maintenance, railway sidings, roads, and telephone and telegraph communication systems, and it has been found advisable in some of these cases to organize a permanent construction force under the control of a trained official of the organization.

The usual owner or engineer of to-day, aside from the great corporations or Governmental organizations, has a great many other things to do than to take the part of a general contractor, and usually he has not the organization or the personnel that the general contractor develops and maintains, with the special knowledge and experience that must be a basis of any attempt to do general contract work. Any owner or engineer who undertakes, seriously, a project in this way without previous experience, and qualified personnel, will have many snares and pitfalls to avoid, and his experience may cost him much from the most unexpected causes. However, the Engineering Profession has given many examples of successful transition into the general contracting business.

It seems well to remember that the engineer is usually charged with more than the design of structures and projects, and the preparation of drawings and specifications. Generally, he is the one who should make the analysis of the situation on which a decision may be based as to the best method of carrying the project from a status of drawings and specifications into a completed plant or structure.

WILSON T. BALLARD,* M. AM. SOC. C. E. (by letter).—The author has discussed a subject of great interest and importance in these days of large construction projects. He announces an effort to throw some light on the subject of the general contract and the segregated contract. He then dismisses the subject of the segregated contract in a few words, by defining it and stating that certain engineering projects, by their nature, are satisfactorily handled by segregated contracts, and proceeds with a detailed discussion of the advantages of the general contract with especial emphasis on its value in building operation. Therefore it seems worth while to bring out in discussion more concerning the segregated contract and, further, to consider certain modifications of the general contract that may broaden its scope of usefulness.

Little, if any, criticism can be offered of the statement that the general contract, in practically every instance, is the better of the two forms for building construction. In the construction of most large buildings a great number of small sub-contractors are employed, many of whom are hardly more than groups of highly specialized laborers whose efforts must be properly timed and co-ordinated.

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There are many kinds of projects, however, outside the field of building construction. One group of great importance includes the building of large bridges. Each big bridge project divides itself naturally into the construction of substructure and superstructure which, except in the case of arch bridges, are totally different operations and are performed by different kinds of organizations. Frequently on a bridge of any size, the contractors on the substructure and superstructure are each competent and financially responsible. If, in such a case, one or the other occupies the position of general contractor, he does not perform a service of appreciable value in supervising the work of the other. Therefore, the owner is best served by carefully drawn separate contracts awarded to contractors of proven capacity and ability, thus avoiding the payment of double profit.

There is one qualification of the foregoing that, when made, brings work such as described back into the hands of a general contractor without calling on the owner to pay a commission or fee and receiving no commensurate value therefor. This qualification requires that the contractor who assumes the work under a general contract modify his commission in such a way that it is not made to apply to the value of work done by a large sub-contractor whose ability and financial responsibility are equal to or greater than his own. For instance, assume that A, a competent company on foundations, takes the general contract and sublets the steel superstructure to B, one of the large steel companies. A's contract provides that he must deliver the piers at a certain time, after which the company furnishing and erecting the steel must do its work. During this period of steel erection, A has little if anything to do and maintains practically no organization on the job. It would be unfair to the owner therefore for him to include in his fee paid him by the owner an appreciable, if any, percentage of the value of the work done by the steel company since such a charge on his part would constitute a duplication of profit paid by the owner. This applies to other than bridge construction where the work, by its nature, divides itself into two or three large and distinctly different operations.

A further point discussed by Mr. Christie with reference to the 3 or 4% charged by the engineer or architect for supervision during construction, might well be considered. Ordinarily, supervision during construction does not contemplate co-ordination of contractors' functions, but rather is furnished in order that the owner through his agent, the engineer, is assured that all work done is according to plans and specifications. This charge, therefore, does not represent a duplication of profit or fee for general supervision, but rather is compensation to the engineer for a distinct and separate service and one that is indispensable, especially where part or all of the money being spent comes from the investing public through the medium of bankers.

Ward P. Christie,* Assoc. M. Am. Soc. C. E. (by letter).—This paper, which set forth the advantages of the general contract to owners, has developed many interesting and valuable observations. The writer feels indebted to all those who took part in the discussion for having contributed their time and constructive thought to this rather spiceless subject. He would like to respond to the courtesy of discussion by acknowledging each individual observation, but that is not feasible.

Some of the discussion indicates that the paper was interpreted to raise an issue between contractors and professional engineers with respect to the two systems of contracting, but this was not its intent. Such issues as do exist were not assumed to involve the professional man as one party, but to rest principally between two groups of commercial organizations. It is not clear to the writer how a professional engineer can be a fixed proponent of one contracting system on all occasions, unless he ceases to remain professional, or begins to commercialize his profession.

In several parts of the discussion resentment cropped out against any thought that engineering ability, *per se,* does not qualify one to manage construction; and this thought might profitably be developed, but there are so many current conceptions of engineering, that the subject becomes hopelessly confused. It appears advisable, therefore, to eliminate this point as far as possible in comparing the two contract systems. This may be done by assuming that any competent engineer is capable of performing the services of a responsible general contractor.

Since it is not practicable to deal with all the views expressed, the writer's closing discussion is confined to five points which appear to be in need of emphasis or further clarification. They are as follows: (1) Judicious Selection of a System; (2) Engineering Service; (3) Construction Service; (4) Liability of Owners; and (5) Ethical Considerations.

Judicious Selection of a System.—In view of the conflict of thought which usually attends a discussion of contracting methods, the clear observations of Professor Lyle are pleasing. He makes the point that construction projects vary in character from those that are obviously suitable for one system to those that are obviously suitable for the other, and that between these extremes are projects which necessitate, in the selection, an individual exercise of judgment.

An example of a project clearly requiring the general contract is the subway job mentioned by Mr. Ridgway. Its many different parts are so closely interlocked and its management so specialized that segregation into separate contracts would place an enormous risk upon the public body. Probably each contractor would be obliged to include a heavy contingent allowance in his bid to cover the hazard of poor co-ordination.

An example of the other extreme involving little hazard to an owner, is a steel bridge, such as that mentioned by Mr. Ballard, for which the work is physically separated into various parts, which are not built simultaneously. On a project such as this no great degree of co-ordinating management is necessary. Each contractor could work without interference from others, and such interlocking as occurred between abutments, superstructure, and roadway surfacing, could be handled without placing material risk upon an owner.

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* Engr., The Associated General Contractors of America, Washington, D. C.

It should not be inferred from these examples which illustrate only the feature of risk to an owner, that a general contract is unsuitable to projects that lack the characteristic of interlocking. Numerous other factors are involved in a judicious selection, and they can only be judged by considering the circumstances which attend the specific project. Some of these factors are as follows:

(*a*) Magnitude of the entire project and its component parts.
(*b*) Availability of equipment and duplication of contractor's outfits.
(*c*) Competitive conditions in the general and specialized contracting fields.
(*d*) Sources of labor and duplication of labor camps.
(*e*) Necessity for speed.
(*f*) Relative prices as shown by actual bids.

If a contract system were selected according to a judicious weighing of these factors, probably the issue between general and separate contracts would disappear. The issue does not usually arise from disagreement as to the need for co-ordinating management, but from disagreement as to who should perform that service.

Some engineers desire to perform it because they believe they can manage the job and save money for client or employer; some have had unsatisfactory service from general contractors; others are disgusted with the way that some contractors treat their sub-contractors; but in a great many cases separate contracts are used because of self-interest of the agency that has the decision. Some one is interested in expanding or solidifying his job, gaining experience to enter business, or performing the general management for a commission.

If the paper did reflect the categorical "no" for separate contracts, to which Professor Lyle objected, it was an unconscious reaction to the categorical "yes" of many who sell their services or hold their positions through that system. His advice against becoming proponents of either system, sound as it is, will conflict with the self-interest of this class. They are undergoing or have completed the transition from profession to business.

Individuals or firms that desire to engage in the business of construction management, without entering the field as contractors, must become proponents of the separate contract system. Their services are as inherently linked with that system, as a general contractor's are linked with the other. Therefore, except in those cases where decision is in the hands of a professional engineer, who is not desirous of managing construction for financial gain or experience, judicious selection of a system with respect to the clients' or employers' interest is not likely to be the rule. It is too closely linked with the question of profit, commission, or salary.

Engineering Service.—A number of observations made in the paper were based on the assumption that those who call themselves engineers are engaged in the practice of a profession. Otherwise, the discussion becomes irrational. If it is actually desired to clarify the separate-contracts issue, rather than confuse it, a distinction must be made between professional engineering and the many business activities that have borrowed its name.

According to Webster a profession is "a calling in which one professes to have acquired some special knowledge used by way either of instructing, guiding, or advising others, or of serving them in some art". This is a definition that seems to fit the men who, in former years, were called engineers. Those who design construction projects and who supervise their building are undoubtedly doing professional work, if the purpose of supervision is to see that a client obtains what he bought. They have no personal interest in recommending a specific material or system of contracts and are not in such a position that their advice to a client or employer could be influenced by any possibility of their financial gain or loss. These appear to be the basic characteristics of a profession.

When this conception of professional engineering is compared with Mr. Dougherty's definition of an engineer, it would seem that the two bear very little relation to each other. The latter does not appear to describe those men who are the backbone of the Engineering Profession. It fits more closely those who have left the practice and have entered into business.

During the last decade many men who were engaged in the profession of engineering have developed themselves along the line of construction management. They are operating either as contractors, in the generally accepted sense, or as construction managers, under the appellation of engineers. Some who are strictly business executives and completely out of touch with matters of the profession, still cling to their earlier title and inflate the definition of an engineer to fit their later accomplishments. Apparently, the title of engineer is a garment of respectability which one is reluctant to lay aside when he emerges from the profession into the supposedly less respectable field of business.

Such juggling of terms, however, should not obscure the fact that engineering as understood by most of the world is a profession which requires its particular type of mind and ability. It is a most honorable and useful vocation, but proficiency in engineering does not of itself equip one for construction management.

The fact that some individuals pass from engineering into business and succeed, should not delude engineers as a class into thinking that every competent member of the profession can do likewise. The field of construction management and that of engineering are vastly far apart, and few individuals are able to master both. The professional engineer, therefore, should move cautiously when he decides to experiment with separate contracts. He may well pause and consider whether it is ethical for him to acquire his business experience at the risk of some one else, for he is risking his client's or employer's money and not his own.

Construction Service.—Two principal types of construction service are furnished by firms calling themselves either engineers or contractors. One is based on the separate contract system and the other on the general contract. Either may include both design and construction, but this has no particular bearing on the nature of the construction function itself. The one based on separate contracts does not include collectible guaranties, but it protects an owner's interests to the extent of the skill and integrity of the constructing firm. The one based on a general contract gives not only the protection of skill and integrity, but also that of financial responsibility. It includes guaranties covering time of completion, claims, and cost of the project as a completed whole.

Performance of either of these services in a proper manner is an honorable vocation, but one should not be recommended to a client or employer when the other will serve his interests better. If an owner has a definite limit to the amount he may invest in construction, and cannot afford to take a chance on an over-run of cost or damages, he should obviously have a service that will absorb the risk of such occurrences. This can be practically provided only by the use of a general contract. On the other hand, if the owner is not limited to a definite expenditure, and is willing to assume the risk of management, then the separate system fits his needs.

At present, separate contract service is extensively identified with firms called engineers and the general contract with firms called contractors, although on occasion either one or both services may be supplied by the same company. The firms themselves are practically alike, except that some of the directing heads were once engineers, while others grew up in the ranks of business. Irrespective of what they call themselves, the successful ones are headed by competent business executives, and not by professional engineers.

Some of these firms offer various types of super-construction service, dressing them up as professional services; but one is only fooling himself if he thinks that construction management is not a business; and he is certainly fooling his client if he represents it as a profession.

The unvarnished fact is that these super-services are composed of two elements: One is design, furnished by professional men; and the other is construction management, performed by business executives. The fact that designers and managers are associated in one firm does not alter the kind or amount of service necessary for a construction project. It does not eliminate either the fee for design or the fee for management. These fees are merely combined as one and paid to a single concern.

The really significant feature of the super-service is that both the mistakes of engineering, and the shortcomings of management "come home to roost on the same perch". When the job does not progress properly, the firm that combines both design and construction has no one to whom it can "pass the buck". This may be a great advantage to an owner, if he is using a general contract, as any loss from mismanagement will then fall on the constructor; but if he is using separate contracts, the super-service avails him nothing. The managing agency does not assume any over-run of expense, and the loss will fall on the owner.

The significance of these circumstances is readily perceived when one learns what has been decided about the owners' responsibility by the Courts.

Liability of Owners.—Legal aspects of the two contracting systems were treated briefly in the paper; but there was practically no discussion of this subject, and it is of most vital concern to owners. The fact does not seem to be generally known that various warranties, which do not concern public or private owners under a general contract, are placed upon them when the work is divided among independent contractors.

On certain types of projects, on which one contractor is sure to complete his work and be out of the way before another is due to begin, this responsibility may be of little consequence, but on other types it becomes a serious matter. If the work of the various contractors must proceed simultaneously, or if the progress of one depends upon the performance of another, the owner has a legal responsibility for synchronizing and co-ordinating their operations. It is his duty, exercised of course through an agent, to see that no one of the independent contractors injures the work or impedes the progress of another. These conditions are implied toward all, and the Courts have held him accountable.

Naturally, if the managing agency is capable and experienced, it can greatly minimize the risk, but no amount of skill or experience can insure that valid damage claims will not arise. Such insurance can be given only by some agency that is financially able and willing to absorb the loss.

A very clear example of how claims originate merely through interference of one contractor with another is found in the case of the Edge Moor Iron Company *vs.* United States.* The Iron Company had a contract for furnishing and installing certain boilers upon foundations which were being built for the Navy Department by another contractor. Each had a separate contract with the Government. The work was not complicated, and there was not much interlocking with respect to time or position; in fact, the project was one requiring a minimum of co-ordination.

The foundations were not completed according to a schedule such that the Iron Company could carry out its work in a manner inferable from its contract, and it was subjected to considerable delay. This delay, which increased the contractor's expense, constituted a breach of contract on the part of the Government, thereby establishing valid grounds for claim. Mere extension of contract time did not compensate for the delay.

In rendering a decision, the Court of Claims pointed out that, although the foundations were not built by Government forces, the Government could not escape liability. It had breached a condition necessarily implied by the separate contracts, that it would co-ordinate the work of all contractors so that each might perform his part as indicated by his contract.

The Edge Moor decision and other similar ones rendered by the U. S. Supreme Court and the Court of Claims demonstrate that separate contracts contain a serious loophole for claims. If any one of the contractors is of the kind that makes his profit out of suing on technicalities, he can cause trouble on almost any job of interlocking parts. When a suit does occur, the owner's only hope of avoiding loss is that of suing his own agent or a delinquent contractor. Possibly the delinquent contractor may have been unavoidably delayed.

In such cases the contractor's defense is very often made secure by some complication of circumstances or by technicalities which involve the engineer.

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* 61 Ct. Cls. 392.

It is not unprecedented for a number of suits to grow out of a single job. Such occurrences would probably be more frequent were it not for the fact that a responsible contractor ordinarily would rather take a loss than go to Court.

There is no practical way to eliminate the hazards of these circumstances through the wording of contracts. It has been tried repeatedly without success. If the provisions are made "tight" enough to block claims that might arise from separation of the work, they are so "tight" that responsible contractors often decline to bid. If they do bid, they must include in their prices an extra allowance for contingencies.

This situation is partly responsible for the development of the general contract. It serves as a sort of contingency trap to catch all the overcharge, short, and damage claims arising from the complexity of the work. Under this contract any judgments obtained by the specialized contractors or others must be satisfied by the general contractor or his surety. They are hazards under the system.

Ethical Considerations.—The paper was characterized by Mr. Muhs as a plaint that "engineers, by a system of segregating contracts, are usurping the functions and encroaching upon the field of the general contractor". This is a surprising reaction, because such plaint as may have been readable between the lines was not for general contractors. It was for those many men who respect and practice the profession of engineering.

General contractors are not fearful that the need for their services will be eliminated by separate contracts. A few more outstanding failures on public buildings, on which inexperienced men are experimenting with this system, will kill it in the public field. If it should become predominant in private work, contractors will merely adopt the system. If necessary they will associate professional engineers with their organizations, and render both design and construction service. There is nothing mysterious about it that gives a monopoly to management agencies called engineers.

What causes contractors to condemn separate contracts is not a loss of business, or even the nature of the system itself; but the misstatements or ignorance by which owners or employers are frequently induced to use the system. Owners are commonly told that separate contracts provide them with a service as complete and as fully protective as that of the general contract; and that they can eliminate the services of a general contractor and save the expense of his service. Such statements are subtle and dangerous misrepresentations, which many owners are unable to detect.

When an owner is willing to assume the risk entailed by separate contracts and accepts the proposition with full knowledge of their character, no one can have any just complaint about their use. It is a matter of importance only to those directly concerned in the transaction. However, if the owner is induced to use the separate system through statements that it places no more risk and responsibility upon him than does the general contract; or, if an engineer, with no experience in construction management, represents himself as competent to replace an experienced contractor, then a question of ethics is clearly involved.

Such practices are misrepresentations of kind and quality that fall into the same category as the misbranding of commercial products. They are no more related to honorable business than are the actions of some contractors who skimp their work or undertake projects which they know they cannot perform.

For a number of years complaint against the methods used in promoting separate contracts has been growing, but, since the profession has taken no great interest in the matter, and since many contractors had hands that were anything but clean, the subject has had very little frank discussion. It is no longer a matter of concern only to the owner and his engineer. It concerns both the business of contracting and the profession of engineering. From the discussion, it is evident that the basic need for analysis is not on the theory of the systems, but on ethical practice for their use. It is not beyond the possibility that the profession is losing its prestige through the aspirations of members toward business, and through the practices of some who have already made the transition.

To any one who may imagine that the paper was merely a plaint of contractors, or that there is no real question of ethics involved in the use of separate contracts, the writer commends a reading of Section 2, Part III, of the Society's Code of Practice.* It is as follows:

> "The Engineer shall call the owner's attention to the fact that under a general contract for a project the Contractor assumes large responsibilities for organization, co-ordination, and management which, under segregated contracts, must be borne by the Owner or Engineer. Further, that in many cases a general contractor, with control of and responsibility for these phases of organization, co-ordination, and management, may secure economies of costs and time in the benefits of which the Owner will share."

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* Manual of Engineering Practice, No. 1, Am. Soc. C. E.

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Paper No. 1725

CROWN STRESSES IN A SKEW ARCH*

By J. Charles Rathbun,† M. Am. Soc. C. E.

With Discussion by Messrs. B. F. Jakobsen, Edward Godfrey, and J. Charles Rathbun

Synopsis

The purpose of this investigation was to devise a method of finding the stresses at the crown of the ring of a cast skew arch model and to check the results obtained by laboratory experiments with those obtained by theoretical calculations. Unfortunately, the opportunity did not present itself to rebuild the apparatus in order to produce better results and as such an opportunity does not appear imminent, the data thus far obtained are presented for consideration. This investigation has grown out of a study made by the writer in 1923-24.‡

The analysis presented at that time was of a purely theoretical nature. The writer was unable to find experiments either refuting or corroborating the work; nor did the discussions of the paper cite any experiments or theory that could be used as a check. The writer hoped to check the various deductions by a continuation of these experiments, but this has not been practicable. A study of the effect of horizontal loads§ is of especial interest as brought out by A. H. Beyer, M. Am. Soc. C. E.

General

In order that the notation used herein and in the Appendix may be more easily understood, Fig. 1 from the 1924 paper is reproduced as the writer's Fig. 1.

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[*] Published in February, 1928, *Proceedings*.

[†] Prof., Civ. Eng., Antioch Coll., Yellow Springs, Ohio.

[‡] "Analysis of the Stresses in the Ring of a Concrete Skew Arch," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 611.

[§] *Loc. cit.*, p. 657, Conclusion (*e*).

In the present experiments, arches cast from plaster of Paris were secured in a framework made for the purpose (see Figs. 2 and 3). One abutment was held as rigid as possible while the other was secured to a steel frame. To this frame were attached six dynamometers or spring balances by means of which the reactions of the free abutment could be obtained. At two positions on the arch ring steel collars were clamped. These positions were considered as the abutments of the arch ring for the purpose of computation. The dynamometers were adjusted as the load was applied, so that the two collars remained in the same position relative to each other throughout the test.

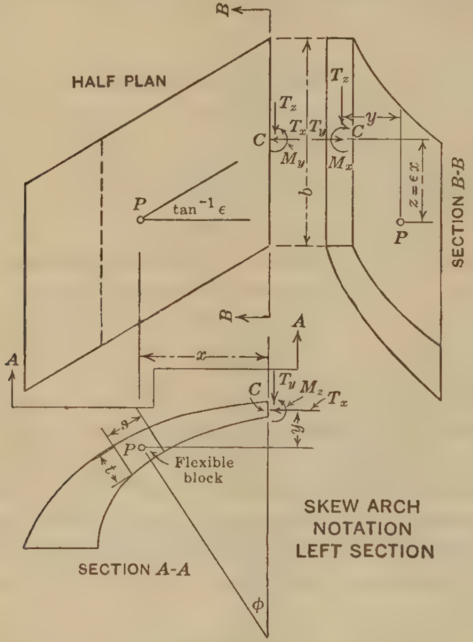

FIG. 1.

Of necessity, in considering the stresses in the ring of a skew arch the magnitude, direction, and location of forces in space must be considered, in contradistinction to forces in a definite plane, as in the right arch theory.

To define the resultant of a system of forces completely requires only six quantities. These may be taken as the components parallel to any three assumed rectangular axes and the moments about these axes. As all these quantities are mutually independent, the six are necessary as well as sufficient, and it is, therefore, important to have six and only six independent dynamometers to determine the location of the force produced by the thrust of the arch ring.

Again, to determine the position of a body in space requires six independent quantities, three to locate the position of a point in it, and three to determine the amount it is rotated about each of three perpendicular axes. These quantities are also independent.

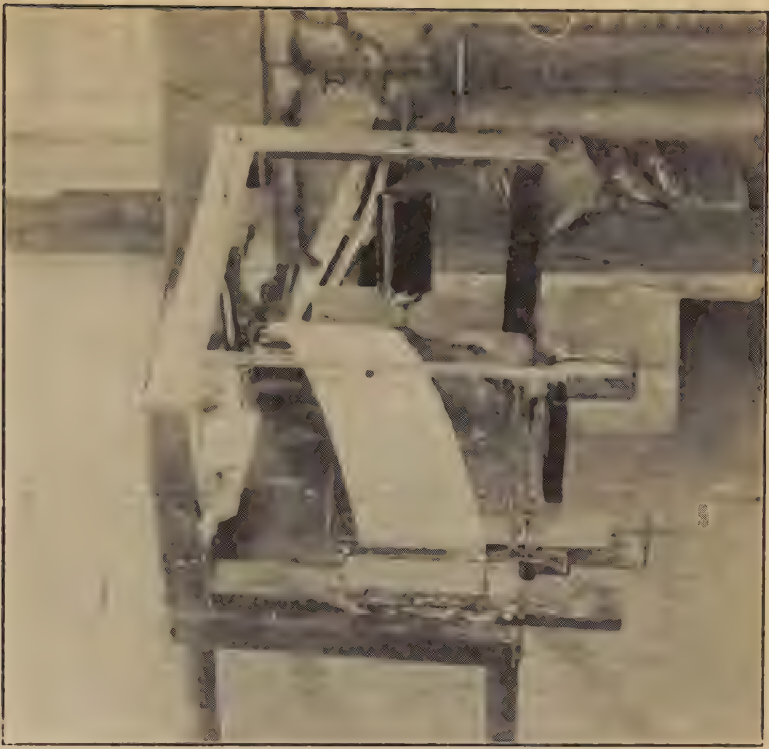

FIG 2 —PLASTER OF PARIS MODEL

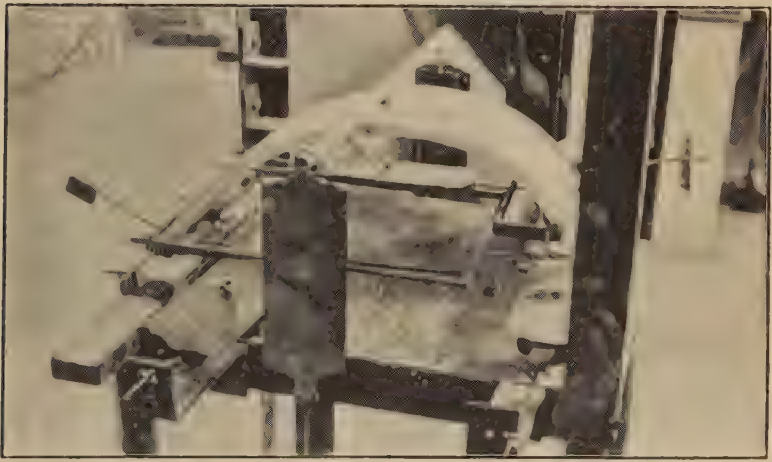

FIG. 3.—VIEW OF PLASTER OF PARIS MODEL.

Therefore, in an experiment of this character, all six components of the abutment reaction must be observed and all six possible movements of the abutment guarded against.

MODELS

The models used were made of plaster of Paris mixed with 60% (by weight) of water. This mix was decided upon after a number of experiments with varying water contents. The value of E was found to vary greatly with the proportion of water used, being about 400 000 lb. per sq. in. when equal parts by weight of water and plaster (100%) were used and 1 100 000 lb. per sq. in. when one-half this quantity of water was taken. Thus, it is quite obvious that great care must be used to secure a uniform mixture. If the mix was too rich the rate of set was rapid and the tendency to lump was considerable. The result, however, was a much stronger material. It was finally decided that the least quantity of water that could safely be used, was 60%, by weight, of the plaster. For the sample of plaster of Paris used in this work this percentage gave ample time to mix the batch, work out all lumps, pour it into the mould, and remove the entrained air before the initial set. At the same time the resulting model was strong enough to withstand rough handling.

A mixture of plaster of Paris and water proved to be a very satisfactory material for the purpose of studying stresses in a model with the view of applying the information thus obtained to a study of stresses in a larger concrete structure. Being a cast material it has somewhat the same characteristics as concrete and when mixed with care is very uniform in quality. After being cast, a model can be sawed or cut with a chisel or plane. This property was very valuable in shaping the skew arch rings.

The ring was cast as the segment of a cylinder with an inside radius of 12 in. and an outside radius of 12¾ in. The form for the intrados consisted of a sheet of galvanized iron held in a semi-circular shape by a wooden frame. Two pieces of ¾ by ¾-in. rubber packing were laid on this form a distance apart that would insure sufficient material for trimming the completed model. Over these was placed a piece of galvanized sheet iron as a form for the extrados. This was fastened by bolts attached to the edges and screwed to braces over the wooden frames. Thus, the two pieces of sheet iron were held securely ¾ in. apart. The form was then turned completely over, and the arch was cast by pouring the soupy mixture in one side and letting it rise to the other side. A wire was threaded through the forms and worked back and forth to remove any air pockets that might be present. The sheet iron was greased to prevent the plaster from sticking to the forms. The arch was allowed to set 24 hours before the forms were removed.

Lines were drawn on the outer sheet of iron with a soft lead pencil. These lines were laid out as a rectangular grid before the sheet was bent over the forms, and the sheet was placed so that one set of these lines was parallel to the axis of the barrel of the arch. The crown and lines parallel to and 60° from it were particularly noted. When the forms were removed the lines were found plainly marked on the model and aided greatly in truing up and setting it.

The rubber packing when laid on the circular form did not lie in a plane, of course, so the model had to be trimmed. This was done by clamping a thin board to the arch so that it was tangent at the crown line and skewed to the axis of the barrel. The model was then laid on a table with the plane of the board perpendicular to the table. Two lines were scribed on the plaster parallel to the table surface. These lines marked the two sides of the completed arch ring. By using an ordinary hand-saw with coarse teeth, the ring was sawed close to these lines and then trimmed down with a carpenter's jack-plane.

Thus, the completed model consisted of an arch ring of a cast material with a low modulus of elasticity and considerable strength. It was formed by the segment of a cylinder subtending an arc of 120° and was limited by two parallel planes, which make an angle of $(90 - \epsilon)$ with the axis of the barrel of the arch, ϵ being the angle of skew.

The lines 60° from the crown were used to set the collars for recording abutment deflections.

APPARATUS

One of the fundamental assumptions in the arch theory is that the relative position of the abutments does not vary. In designing the testing apparatus it was necessary, therefore, to be assured of this condition and, at the same time, to be able to record the six components of the abutment reaction.

The arch ring proper was considered to lie between the two collars that were clamped to the arch ring. (See Figs, 2, 3, and 4.) Each of these collars had contact with the ring at four points by means of sharp screws resting against fiber washers. The washers rested on the arch ring at the centers of the four sides of a section of the ring. In order to be sure that the collars maintained the same position relative to each other, six independent points of contact had to be maintained. This was done by means of a system of levers. The left collar had attached thereto an arm that reached to within $\frac{5}{8}$ in. of another arm attached to the right collar. In this space was inserted a steel phonograph needle sharpened at both ends. To the needle was attached a stiff wire pointer, properly counterweighted so that the center of gravity lay at the center of the needle. The end of the pointer was in line with the axis of the needle. A short wire was attached to the nearest collar and the free end was adjusted so as to mark the initial position of the end of the pointer. By this device a movement, either vertical or horizontal, of one end of the needle relative to the other, was multiplied $\frac{17 \times 8}{5}$, or 27.2 times. To the right collar were attached two arms which also engaged phonograph needles in a similar manner. These two needles were placed so that their axes were in line. Thus, there were three needles, each recording two motions, making a total of six independent points of contact between the two collars. To show that they were independent it is only necessary to demonstrate that each needle could be moved either vertically or horizontally without affecting any of the other five settings.

A rotation of the right abutment relative to the left about a line joining the points, farthest from their respective pointers, of Needles Nos. 2 and 3 would give a vertical movement only to the pointer of Needle No. 1 without affecting the setting of the other two pointers. A result similar to that obtained for Needle No. 1 could be obtained for Needles Nos. 2 and 3. A horizontal rotation of the right abutment relative to the left about the vertical line formed by the intersection of the planes perpendicular to Needles Nos. 2 and 3 and passing through the one of their points in contact with the right abutment, would give a horizontal movement to the pointer of Needle No. 1. Similarly, for Needles Nos. 2 and 3. For Needle No. 3, it is to be noted that the axis of rotation is at infinity. This does not affect the result.

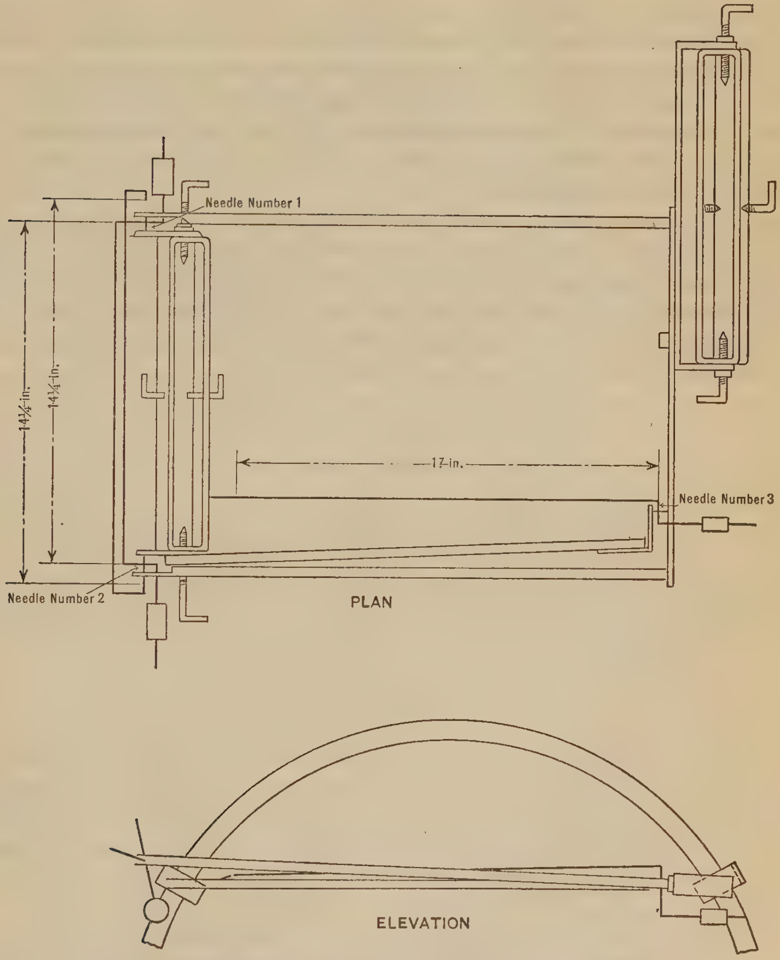

Fig. 4.—Device for Recording Movements of Abutment.

The load used was a concentrated load placed at the center of the crown. The method of loading is shown in Figs. 2 and 3. A ½-in. cut washer was placed on the arch ring, and on this was placed a ½-in. nut, in which was a stud with the end rounded off. The pipe that was used as a beam to apply the load was flattened by filing slightly where it rested on the stud. The ends of the pipe carried milk scales for dynamometers. The load was applied by inducing a force on the scales with a hook-bolt and butterfly nut. (See lower right-hand corner of Fig. 3 for this detail.)

The left abutment, Fig. 3, was placed in a steel trough and plaster of Paris was poured around it. In doing this, care was taken that the collar was free from the plaster or trough. The trough was bolted securely to a steel table made for this investigation. To the table was welded a very rigid steel frame made of angle iron. Although this property was a convenience, it was not a necessity.

In like manner, the right abutment was secured to a steel trough which was connected to the table by six dynamometers, *A*, *B*, *C*, *D*, *E*, and *F*, the first three being horizontal, and the last three vertical, as shown in Fig. 5. These six dynamometers were arranged so that they were independent of each other, for it is easy to conceive of a force that will affect any one of them without affecting the others.

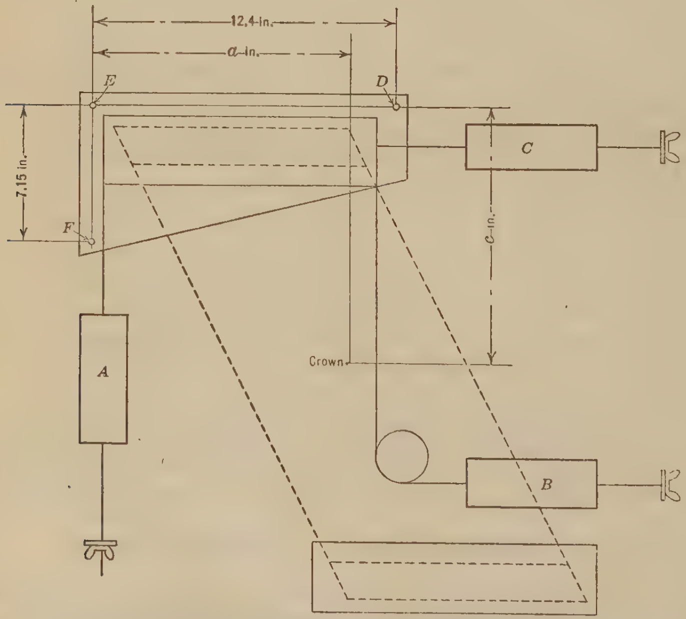

Fig. 5.—Location of Dynamometers with Respect to Crown of Arch.

The scales were adjusted after each increment of load was added until each of the three pointers was at its initial position; therefore, the collars remained in the same relative position to each other throughout the experiment and any deflection due to lack of rigidity of the table, frame, troughs, or arch ring from trough to collar, was taken up while adjusting the scales.

METHOD OF PERFORMING EXPERIMENT

After the model was trued up, as previously described, the collars were placed in position on the marks drawn for them. The free trough was then blocked in place on the table, at the same elevation as the fixed trough and in a position that allowed the vertical scales to hang plumb. It was clamped securely and the model was placed in the troughs, leveled at the crown, and oriented so that the barrel of the arch was parallel to the plane through the axes of Scales *D* and *E*. The portions of the arch ring in the troughs were thoroughly wetted, after which plaster of Paris, mixed with 60% of water, was poured around them until the troughs were filled. When placed thus, the arch had no initial stress, except that due to its own weight. After the plaster had set 24 hours the needles were placed in position and adjusted. The loading beam was then placed across the top of the arch and the dynamometers were hung therefrom. Enough load was placed on the dynamometers to hold them steady. The load then on the crown was less than 10 lb. The dynamometers at the right abutment were tightened and the clamps removed. They were then adjusted until all the needle pointers were back to their initial position, both vertically and horizontally. This was taken as the zero crown load in constructing Figs. 8 to 12.

The crown load was added by small increments as shown in Tables 1 to 6, inclusive, and, after each increment, the dynamometers were re-adjusted. This was continued until the capacity of one of the dynamometers was reached, or the crown load exceeded its initial value by 50 lb.

Although the re-adjustment of the needles by applying loads to the dynamometers was in the nature of solving six equations in six unknowns by the cut and try method, after a little practice it could be done in a few minutes.

PRELIMINARY EXPERIMENT

After the preliminary work to ascertain the most satisfactory mix for plaster of Paris, an experiment was considered necessary to gain an idea of the maximum vertical crown load that could be placed on the model without injury. From these data the testing apparatus was designed, or more particularly, the capacity required for the dynamometers was ascertained.

To obtain this information an arch was cast, the ends were placed in notches cut in a 2-in. board and were plastered into place. (See Fig. 6.) The board and arch were put on a platform scale and a pipe was placed on the crown parallel to the axis of the barrel of the arch. A load was applied to this pipe by means of a plank resting on a knife-edge and holding a vessel on one end and a counterweight on the other. The knife-edge was off the scale and the load was applied by pouring water in the vessel, thus tipping the plank on to the arch. It was measured by weighing the added load on the scales. The deflections were taken at the crown with an Ames dial. In this arch the angle of skew was 22½° and the breadth measured parallel to the abutments was 10 in. Fig. 7 shows the arch after failure. Note that the breaks are parallel to the axis of the barrel. The results of this run are given in Fig. 8.

Comparison with Theory

The components of the crown reactions on a skew arch ring of the character of the models used have been determined for a vertical crown load using the elastic theory previously mentioned. This work is given in the Appendix. By taking the readings of the abutment dynamometers, as determined from Tables 1, 2, and 3, and from these, computing the stresses at the crown, a comparison can be made.

The crown reactions were computed from the following equations of equilibrium for the left half of the arch ring:

$$\left.\begin{aligned} T_x &= A + B \\ T_y &= D + E + F \\ T_z &= C \\ M_z &= c\,T_y - 7.15\,F - e\,T_x \\ M_x &= 12.4\,D - a\,T_y + d\,T_z \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, A, B, C, D, E, and F are the abutment dynamometer readings, and a = crown to plane, $E\,F$; b = width of arch ring; c = crown to plane, $D\,E$; d = elevation of crown above C; and e = elevation of crown above plane, $A\,B$, as defined in Tables 1, 2, and 3. The values of the dynamometer readings were taken from the slopes of the lines as plotted from these tables.

Fig. 9 was plotted from Table 1. A 50-lb. increase in the crown load would produce the following increases in the abutment dynamometer readings: A = 26.5 lb.; B = 15.5 lb.; C = 19.5 lb.; D = 10.3 lb.; E = 21.3 lb.; F = — 3.3 lb.; which with the following measurements: a = 11.125 in.; c = 15.00 in.; d = 8.31 in.; and e = 8.16 in., substituted in Equation (1), give T_x = 42.0 lb.; T_y = 27.3 lb.; T_z = 19.5 lb.; M_x = — 21.7 lb.; and M_z = 85.8 lb.

The theoretical values are: T_x = 40.4 lb.; T_y = 25.0 lb.; T_z = 15.7 lb.; M_x = — 18.1 lb.; and M_z = 61.2 lb.

Fig. 10 was plotted from Table 2. A distinct break occurs in the curves at the 30.5-lb. load, where Dynamometer F ran to zero and it became necessary to place a weight on the trough where it was attached to this dynamometer.

From the lower part of the curves for a 50-lb. increase: A = 12.4 lb.; B = 25.9 lb.; C = 21.0 lb.; D = 13.5 lb.; E = 18.2 lb.; and F = — 7.9 lb.

With a = 16.6 in ; c = 14.85 in.; d = 9.5 in.; and e = 9.34 in., Equations (1) gives, T_x = 38.3 lb.; T_y = 23.8 lb.; T_z = 21.0 lb.; M_x = — 28.2 lb.; and M_z = 51.2 lb.

From the upper part of the curves for a 50-lb. increase: A = 17.2 lb.; B = 22.3 lb.; C = 26.0 lb.; D = 11.4 lb.; E = 21.5 lb.; and F = — 7.5 lb.; which gives as values for the crown forces and moments: T_x = 39.5 lb.; T_y = 25.4 lb.; T_z = 26.0 lb.; M_x = — 33.3 lb.; and M_z = 65.3 lb.

The theoretical values are: T_x = 40.6 lb.; T_y = 25.0 lb.; T_z = 23.8 lb.; M_x = — 34.6 lb.; and M_z = 60.8 lb.

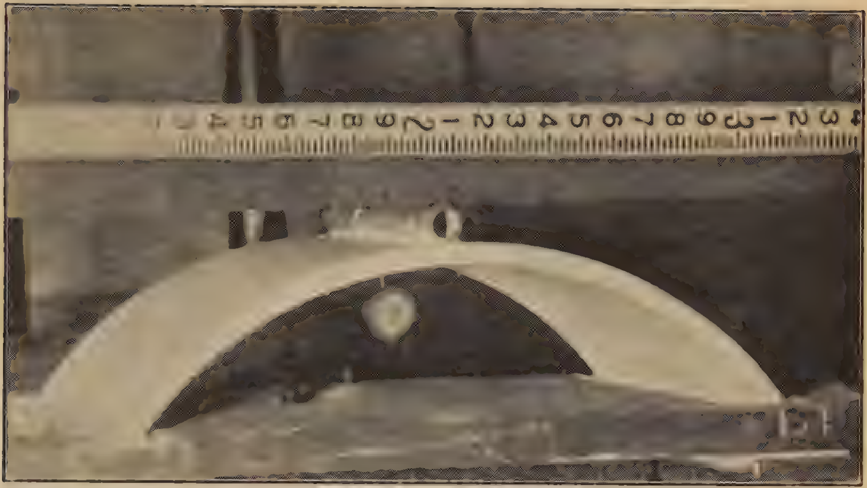

FIG. 6.—TEST ON UNLOADED ARCH.

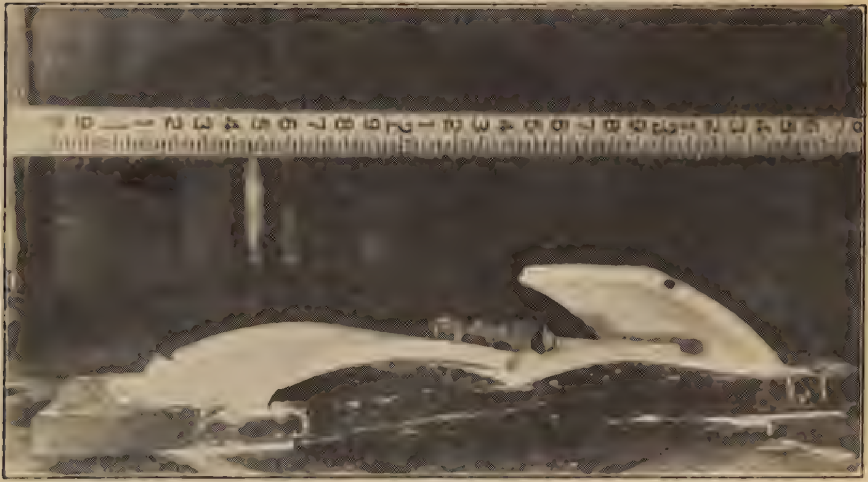

FIG. 7.—VIEW OF ARCH AFTER FAILURE.

Fig. 11 was plotted from Table 3. A 50-lb. increase in the crown load would produce the following: $A = 16.0$ lb.; $B = 27.2$ lb.; $C = 27.6$ lb.; $D = 15.0$ lb.; $E = 16.2$ lb.; and $F = -2.4$ lb. With $a = 17.0$ in.; $c = 15.1$ in.; $d = 9.26$ in.; and $e = 9.1$ in., Equations (1) gives: $T_x = 43.2$ lb.; $T_y = 28.8$ lb.; $T_z = 27.6$ lb.; $M_x = -48.0$ lb.; and $M_z = 58.9$ lb.

The theoretical values are: $T_x = 40.8$ lb.; $T_y = 25.0$ lb.; $T_z = 31.0$ lb.; $M_x = -43.3$ lb.; and $M_z = 59.8$ lb.

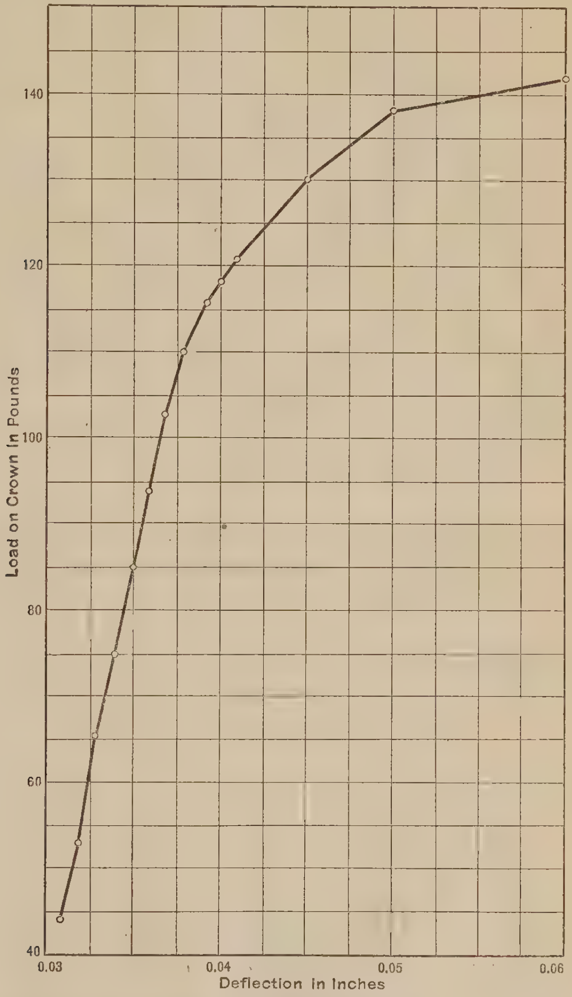

FIG. 8.—PRELIMINARY RUN.

Although the agreement of the theoretical and the observed values is not as close as could be desired, considering the many sources of error in the experiments, it is close enough to convince one that the theory is a reliable method of obtaining stresses in a skew arch ring, and the writer feels that

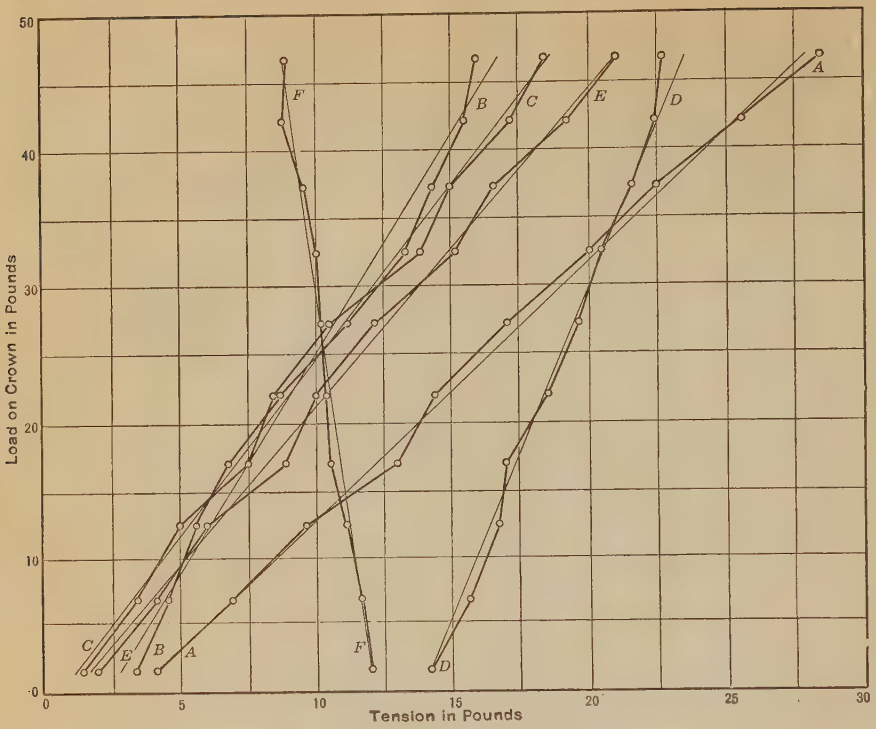

FIG. 9.—RUN NO. 1, ARCH NO. 1.

with apparatus of sufficient refinement the agreement would be closer. This work, however, would involve the expenditure of a greater amount of money and time than the writer has been able to obtain.

TABLE 1.—DATA FROM RUN NO. 1, ARCH NO. 1.

Crown load.	POINTS ON THE ARCH RING:					
	A.	*B.*	*C.*	*D.*	*E.*	*F.*
1.7	4.1	3.2	1.3	14.2	1.9	12.0
7.0	6.8	4.4	3.4	15.5	4.1	11.6
12.0	9.6	5.6	5.0	16.6	5.9	11.1
17.1	13.0	6.7	7.5	16.9	8.9	10.5
22.0	14.3	8.7	8.4	18.4	9.9	10.3
27.4	16.9	11.4	10.4	19.6	12.2	10.2
32.6	20.1	13.3	13.8	20.4	15.2	10.0
37.4	22.4	14.2	14.9	21.6	16.5	9.6
42.3	25.5	15.4	17.2	22.3	19.1	8.8
46.9	28.3	15.8	19.4	22.7	20.9	9.0

NOTE.—a = 11.125 in.; b = 9.75 in.; c = 15.00 in.; d = 8.31 in.; e = 8.16 in.; and angle of skew = 22 degrees.

EFFECT OF THE ABUTMENT MOVEMENTS

The abutment of the arch with a 31° skew was moved in a horizontal direction without rotation parallel to the axis of the barrel of the arch, in order to test the effect on the abutment reactions of such a movement. Unfortunately, no readings were taken to ascertain the amount of this movement. Needles Nos. 1 and 2 were kept in position while the pointer of Needle No. 3 was moved horizontally by varying the load on the dynamometers. Rotation of the abutments was also prevented. From Table 4 and Fig. 12 the tendency can be easily noted. As Dynamometer *C* was released in order to keep the span of the arch a constant, Dynamometer *B* received load from Dynamometer *A*, while Dynamometer *D* received load from Dynamometers *E* and *F*.

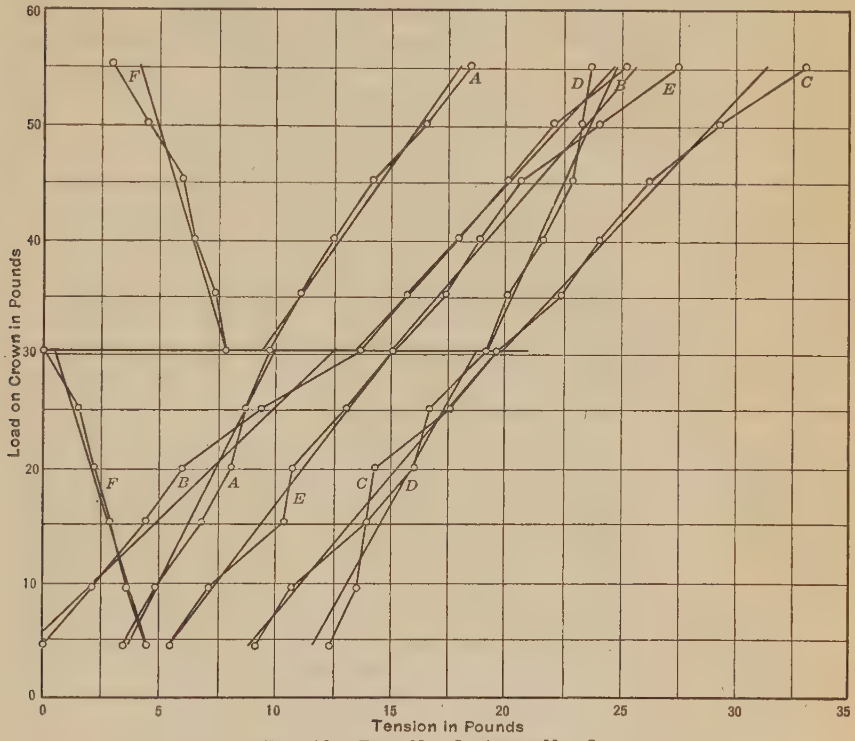

FIG. 10.—RUN NO. 2, ARCH NO. 2.

TABLE 2.—DATA FROM RUN NO. 2, ARCH NO. 2.

Crown load.	*A.*	*B.*	*C.*	*D.*	*E.*	*F.*
4.6	3.5	—0.2	9.1	12.3	5.5	4.5
9.85	4.85	2.1	10.75	13.55	7.15	3.6
15.5	6.9	4.4	14.1	14.05	10.4	2.8
20.2	8.1	5.9	14.3	16.0	10.7	2.1
25.3	8.7	9.5	17.5	16.7	13.1	1.5
30.5	9.8	14.0	19.6	19.2	15.2	0.0
30.5	9.8	14.0	19.6	19.2	15.2	7.9
35.3	11.1	15.7	22.3	20.0	17.3	7.3
40.2	12.5	17.9	24.0	21.6	18.8	6.4
45.4	14.2	20.0	26.1	22.8	20.5	6.0
50.4	16.4	22.0	29.3	23.2	23.9	4.5
55.3	18.4	25.0	32.9	23.6	27.3	3.0

NOTE.—a = 16.60 in.; b = 8 in.; c = 14.85 in.; d = 9.50 in.; e = 9.34 in.; and angle of skew = 31 degrees.

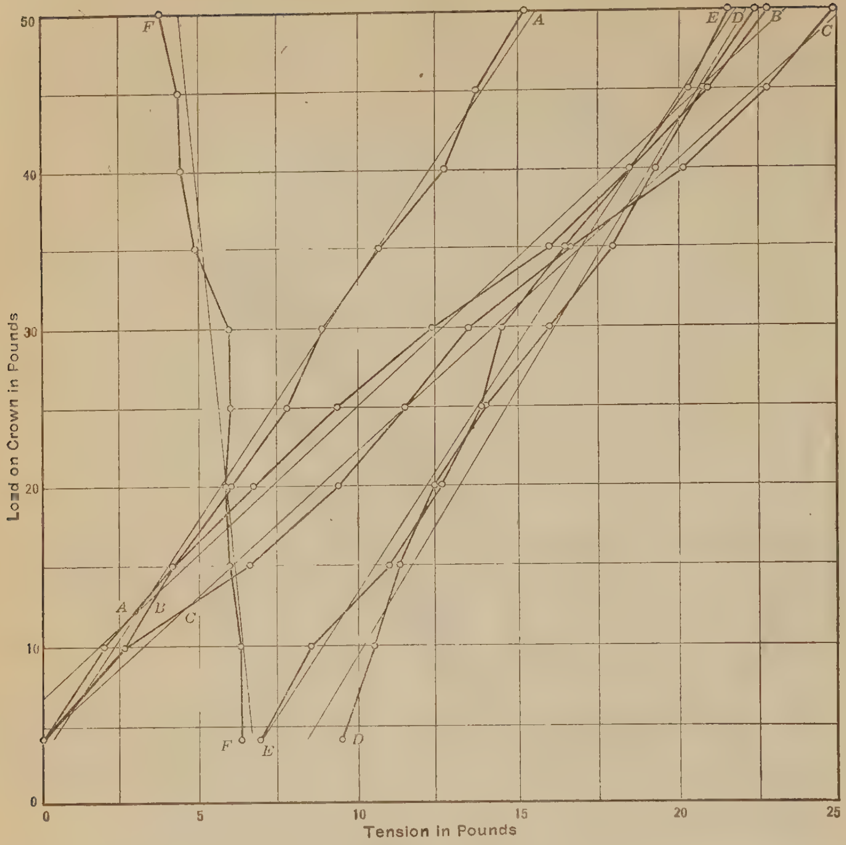

FIG. 11.—RUN NO. 3, ARCH NO. 3.

TABLE 3.—DATA FROM RUN NO. 3, ARCH NO. 3.

Crown load.	*A.*	*B.*	*C.*	*D.*	*E.*	*F.*
4	0.0	0.0	0.0	9.5	6.9	6.3
10	2.0	2.7	2.7	10.5	8.6	6.3
15	4.2	4.2	6.5	11.3	11.0	5.9
20	6.0	6.7	9.4	12.5	12.6	5.9
25	7.8	9.4	11.5	14.0	13.7	6.0
30	9.0	12.3	13.5	16.0	14.6	5.9
35	10.7	16.0	16.7	18.0	16.6	4.9
40	12.7	18.6	20.2	19.3	18.6	4.4
45	13.8	21.0	22.8	20.7	20.3	4.4
50	15.3	22.8	24.9	22.5	21.4	3.8

NOTE.—a = 17.00 in.; b = 9 in.; c = 15.10 in.; d = 9.26 in.; e = 9.10 in.; and angle of skew = 37 degrees.

A third arch was put in the testing frame and a load of about 30 lb. was placed on the arch crown. This arch had an angle of skew of 37° and a length of abutment of 9 in. The abutment was moved parallel to itself without rotation and in a direction to widen the span, horizontally but at right angles to the barrel of the arch. Table 4 and Fig. 12 give the effect of this movement. This is a possible direction in which the arch may slip under load. It is interesting to note that the horizontal force, *B*, becomes zero, or that the abutment force is very near the point of the arch. Table 5 is produced by a movement similar to the one producing Table 3; that is, parallel to the axis of the barrel of the arch. It is interesting to compare the results of these two movements.

TABLE 4.—DATA FROM RUN NO. 4, ARCH NO. 2.*

Crown load.	*C.*	*A.*	*B.*	*D.*	*E.*	*F.*
50	16.5	7.2	23.2	21.8	17.0	5.0
50	14.0	4.5	25.2	24.0	15.8	4.0
50	12.0	2.5	28.2	26.0	13.8	3.7
50	9.0	1.0	31.3	28.1	11.5	3.2
50	6.0	0.0	34.3	30.2	10.0	2.5
550	3.0	—1.2	37.2	32.6	8.3	2.2
50	0.0	—2.2	39.1	35.1	6.1	1.7

* Abutment moved horizontally and parallel to barrel of arch.

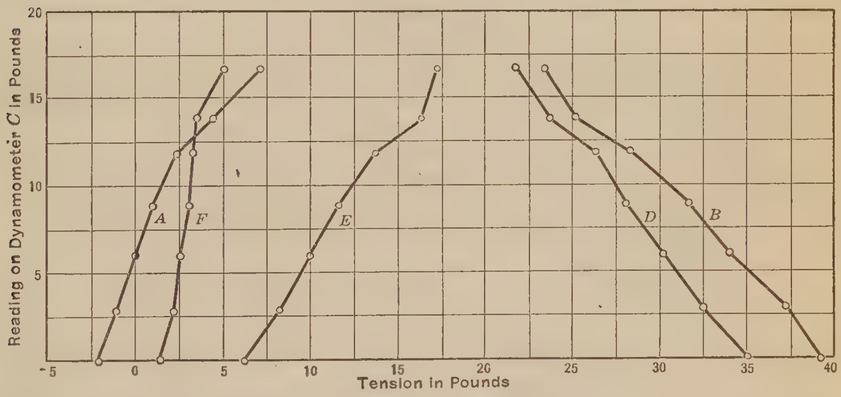

FIG. 12.—RUN NO. 4, ARCH NO. 2.

TABLE 5.—DATA FROM RUN NO. 5, ARCH NO. 3.*

Crown load.	*A.*	*B.*	*C.*	*D.*	*E.*	*F.*
30.2	9.1	6.3	8.8	16.6	13.2	4.9
30.0	11.4	0.0	6.6	15.1	12.7	5.0

* Abutment moved horizontally and perpendicular to barrel of arch.

CONCLUSIONS

Although the apparatus for recording abutment movements could be improved a great deal by being made lighter and a method could be devised for controlling the pressure on the needles, it is the writer's opinion that the following conclusions are warranted:

(1) If the abutment moves in a known direction and amount, the stresses are redistributed in a manner that can be foreseen from the elastic theory.* They may cause the heavier load on one or the other end of the abutment, depending on the direction of slipping.

(2) The force represented by T_z, or Dynamometer C, must be provided for in the design of a skew arch ring.

(3) The obtuse corner of the abutment does not necessarily carry any heavier load than the acute corner, and may carry less.

(4) The discrepancies between the values obtained by the arch theory and those obtained by experimentation are within the range of experimental error so that this set of experiments supports the theory.

ACKNOWLEDGMENT

Appreciation is extended to C. E. Magnusson, M. Am. Soc. C. E., for obtaining funds from the University of Washington Experiment Station with which to make the necessary apparatus and models to conduct these tests; and, also, to W. R. Engstrom, Jun. Am. Soc. C. E., for obtaining a large percentage of the data herein given.

APPENDIX

The theoretical values for the crown reactions for a concentrated vertical central load are given by solving the following six equations:†

$$\begin{aligned}\left(\int y^2 \frac{d\,s}{I_z} + \varepsilon^2 \lambda \int x^2 \sin^2 \phi \frac{d\,s}{F}\right) T_x - \left(\varepsilon \lambda \int u\, x \sin \phi \frac{d\,s}{F}\right) T_z \\ + \left(\int y \frac{d\,s}{I_z}\right) M_z - \left(\varepsilon\,\lambda \int x \sin \phi \cos \phi \frac{d\,s}{F}\right) M_x \\ = \frac{1}{2}\int x\,y \frac{d\,s}{I_z} W + \frac{1}{2}\varepsilon^2 \lambda \int x^2 \sin \phi \cos \phi \frac{d\,s}{F} W \ldots\ldots (2a)\end{aligned}$$

$$\begin{aligned}\left(\int x^2 \frac{d\,s}{I_z} + \varepsilon^2 \lambda \int x^2 \cos^2 \phi \frac{d\,s}{F}\right) T_y - \left(\varepsilon \lambda \int x \sin \phi \cos \phi \frac{d\,s}{F}\right) M_y \\ = -\frac{1}{2}\int x^2 \frac{d\,s}{I_z} W - \frac{1}{2}\varepsilon^2 \lambda \int x^2 \cos^2 \phi \frac{d\,s}{F} W \ldots\ldots\ldots (2b)\end{aligned}$$

$$\begin{aligned}\left(-\varepsilon \lambda \int u\, x \sin \phi \frac{d\,s}{F}\right) T_x + \left(\lambda \int u^2 \frac{d\,s}{F}\right) T_z + \left(\lambda \int u \cos \phi \frac{d\,s}{F}\right) M_x \\ = -\frac{1}{2}\varepsilon\,\lambda \int u\, x \cos \phi \frac{d\,s}{F} W \ldots\ldots\ldots\ldots\ldots\ldots (2c)\end{aligned}$$

$$\left(\int y \frac{d\,s}{I_z}\right) T_x + \left(\int \frac{d\,s}{I_z}\right) M_z = \frac{1}{2}\int x \frac{d\,s}{I_z} W \ldots\ldots\ldots (2d)$$

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* Compare the results of Tables 4 and 5 with the statements in *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 675.

† *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 617–618 and pp. 643–644.

$$\left(-\varepsilon\,\lambda\int x\sin\phi\cos\phi\,\frac{d\,s}{F}\right)T_y+\left(\lambda\int\sin^2\phi\,\frac{d\,s}{F}\right)M_y$$

$$=\frac{1}{2}\,\varepsilon\,\lambda\int x\sin\phi\cos\phi\,\frac{d\,s}{F}\,W\ldots\ldots\ldots\ldots\ldots\ldots(2e)$$

$$\left(-\varepsilon\,\lambda\int x\sin\phi\cos\phi\,\frac{d\,s}{F}\right)T_x+\left(\lambda\int u\cos\phi\,\frac{d\,s}{F}\right)T_z$$

$$+\left(\lambda\int\cos^2\phi\,\frac{d\,s}{F}\right)M_x=-\frac{1}{2}\,\varepsilon\,\lambda\int x\cos^2\phi\,\frac{d\,s}{F}\,W\ldots\ldots(2f)$$

The solution of Equations (2*b*) and (2*e*) is obviously $T_y=-\frac{1}{2}$ and $M_y=0$, and no further consideration of these equations is necessary.

As the arch consists of the segment of a circular cylinder, from the properties of a circle the following substitutions may be made:

$$x=r\sin\phi$$

$$y=r\,(1-\cos\phi)$$

$$u=y=r\,(1-\cos\phi)$$

$$v=x=r\sin\phi$$

$$d\,s=r\,d\,\phi$$

$$\frac{1}{I_x}=\frac{12}{b\,d^3}=4\times\frac{3}{b\,d^3}$$

$$\frac{\lambda}{F}=\frac{9}{b\,d^3}=3\times\frac{3}{b\,d^3}$$

b = width of arch ring.

d = thickness of arch ring.

After dividing through by the common term, $\frac{3\,r^2}{b\,d^3}$, or $\frac{3\,r}{b\,d^3}$, this reduces the equations to:

$$\left(4\int(1-\cos\phi)^2\,d\,\phi+3\,\varepsilon^2\int\sin^4\phi\,d\,\phi\right)r\,T_x$$

$$-\left(3\,\varepsilon\int(1-\cos\phi)\sin^2\phi\,d\,\phi\right)r\,T_z+\left(4\int(1-\cos\phi)\,d\,\phi\right)M_z$$

$$-\left(3\,\varepsilon\int\sin^2\phi\cos\phi\,d\,\phi\right)M_x=2\,r\int(1-\cos\phi)\sin\phi\,d\,\phi$$

$$+\frac{3}{2}\,\varepsilon^2\,r\int\sin^3\phi\cos\phi\,d\,\phi\ldots\ldots\ldots\ldots\ldots\ldots(3a)$$

$$\left(-3\,\varepsilon\int(1-\cos\phi)\sin^2\phi\,d\,\phi\right)r\,T_x+\left(3\int(1-\cos\phi)^2\,d\,\phi\right)r\,T_z$$

$$+\left(3\int(1-\cos\phi)\cos\phi\,d\,\phi\right)M_x$$

$$=-\frac{3}{2}\,\varepsilon\,r\int(1-\cos\phi)\sin\phi\cos\phi\,d\,\phi\ldots\ldots\ldots\ldots(3c)$$

$$\left(4\int(1-\cos\phi)\,d\,\phi\right)r\,T_x+\left(4\int d\,\phi\right)M_z=2\,r\int\sin\phi\,d\,\phi\ldots(3d)$$

$$\left(-3\,\varepsilon\int\sin^2\phi\cos\phi\,d\,\phi\right)r\,T_x+\left(3\int(1-\cos\phi)\cos\phi\,d\,\phi\right)r\,T_z$$

$$+\left(3\int\cos^2\phi\,d\,\phi\right)M_x=-\frac{3}{2}\,\varepsilon\,r\int\sin\phi\cos^2\phi\,d\,\phi\ldots\ldots(3f)$$

Integrating and substituting the limits of $\frac{\pi}{3}$ and 0, and reducing terms:

$$\left(\frac{4\pi}{3} - 4\sqrt{3} + \frac{\sqrt{3}}{2} + \frac{2\pi}{3} - \frac{9\sqrt{3}}{64}\varepsilon^2 - \frac{9\sqrt{3}}{32}\varepsilon^2 + \frac{3\pi}{8}\varepsilon^2\right) r\,T_x + \left(\frac{3\sqrt{3}}{8}\varepsilon - \frac{\pi}{2}\varepsilon + \frac{3\sqrt{3}}{8}\varepsilon\right) r\,T_z + \left(\frac{4\pi}{3} - \frac{4\sqrt{3}}{2}\right) M_z - \left(\frac{3\sqrt{3}}{8}\right)\varepsilon\,M_x = -\left(2r + \frac{3}{2}r - \frac{27}{64}\varepsilon^2 r - 4r\right)\frac{1}{2} \quad \ldots\ldots (4a)$$

$$\left(\frac{3\sqrt{3}}{8} - \frac{\pi}{2}\varepsilon + \frac{3\sqrt{3}}{8}\varepsilon\right) r\,T_x + \left(\pi - 3\sqrt{3} + \frac{3\sqrt{3}}{8} + \frac{\pi}{2}\right) r\,T_z + \left(\frac{3\sqrt{3}}{2} - \frac{3\sqrt{3}}{8} - \frac{\pi}{2}\right) M_x = -\left(\frac{9}{8}\varepsilon r + \frac{1}{8}\varepsilon r - \varepsilon r\right)\frac{1}{2} \quad \ldots (4c)$$

$$\left(\frac{4\pi}{3} - 2\sqrt{3}\right) r\,T_x + \frac{4\pi}{3} M_z = (-2r + 4r)\frac{1}{2} \quad \ldots\ldots\ldots (4d)$$

$$\left(-\frac{3\sqrt{3}}{8}\right)\varepsilon\,r\,T_x + \left(\frac{3\sqrt{3}}{2} - \frac{3\sqrt{3}}{8} - \frac{\pi}{2}\right) r\,T_z + \left(\frac{3\sqrt{3}}{8} + \frac{\pi}{2}\right) M_x = \left(\frac{1}{8}\varepsilon r - \varepsilon r\right)\frac{1}{2} \quad \ldots\ldots (4f)$$

Substituting the numerical values, these equations reduce to:

$$(0.2210 + 0.4474\,\varepsilon^2)\,r\,T_x - 0.2718\,\varepsilon\,r\,T_z + 0.7247\,M_z - 0.6495\,\varepsilon\,M_x = (0.2500 + 0.2110\,\varepsilon^2)\,r \quad \ldots\ldots (5a)$$

$$-0.2718\,\varepsilon\,r\,T_x + 0.1658\,r\,T_z + 0.3777\,M_x = -0.1250\,\varepsilon\,r \quad \ldots\ldots (5c)$$

$$0.7247\,r\,T_x + 4.1888\,M_z = 1.0000\,r \quad \ldots\ldots (5d)$$

$$-0.6495\,\varepsilon\,r\,T_x + 0.3777\,r\,T_z + 2.2203\,M_x = -0.4375\,\varepsilon\,r \quad \ldots (5f)$$

the solutions of which are:

$$2\,T_x = \frac{2.0217 + 0.0689\varepsilon^2}{1.2550 + 0.0148\varepsilon^2}$$

$$2\,T_z = \frac{1.9620 + 0.0948\varepsilon^2}{1.2550 + 0.0148\varepsilon^2}\,\varepsilon$$

$$2\,M_x = \frac{-0.2340 - 0.0017\varepsilon^2}{1.2550 + 0.0148\varepsilon^2}\,\varepsilon\,r$$

$$2\,M_z = \frac{0.2495 - 0.0048\varepsilon^2}{1.2550 + 0.0148\varepsilon^2}\,r$$

TABLE 6.—ARCH DATA FOR $r = 12\frac{3}{4}$ INCHES.

Skew angle.	22°	81°	37°
ϵ	0.404	0.600	0.754
T_x	0.808	0.812	0.816
T_y	0.314	0.475	0.619
M_x	—0.361	—0.691	—0.866
M_z	1.224	1.216	1.197

With $r = 12\frac{3}{4}$ in., these values, for a unit central load, are as given in Table 6.

DISCUSSION

B. F. JAKOBSEN,[*] M. AM. SOC. C. E. (by letter).—The writer is interested in arches as applied to dams and has studied Professor Rathbun's paper with great interest. Bearing in mind the complicated stress distribution in the structure and the relatively simple testing apparatus, the agreement between the experimental values and the calculated values based on Professor Rathbun's theory,[†] seems very good; especially when account is taken of the fact that in order to simplify the mathematics some approximations must be made in the assumptions, and that the measurements are not exact.

Assuming that the mathematical deductions are correct, the tests show that the fundamental assumptions made by Professor Rathbun lead to reliable results. The way is now open for the advocates of "horse sense"[‡] to test their "theories" and submit the results to the profession. These theories, based on "horse sense," are likely to have the advantage of simplicity and the disadvantage of being incorrect, as may be inferred from the number of failures recorded.[§] An exact theory cannot be obtained for the reason that the modulus of elasticity is not the same for all parts of the structure and varies with time; the deformations continue to increase with time even when the load is constant,[‖] temperatures vary, etc.

The development of a useful theory depends largely on the ability and skill of the scientist to distinguish the essential from the unimportant and to formulate such assumptions as will lead to reliable results without undue mathematical complications.

The writer is especially interested in Fig. 8 because it shows the great and rapidly increasing deflection of the arch as the breaking load is approached. When the load was increased from zero to about 102 lb., the arch deflected about 0.01 in., while, when the load was increased from 102 lb. to 142 lb. (the breaking load), the additional deflection was 0.023 in. This means that, in the sections subjected to bending stresses, the neutral axis was shifted toward the compression side and the section was able to carry a much larger bending moment than calculations based on the tensile strength of the material would indicate.

Professor Bach[¶] has made tests on cast iron in bending and, at the same time, has tested the cast iron in tension and compression in order to obtain the stress-strain curve. By this means he found that the bending moment necessary to bring about failure was about 1.66 times greater than the theoretical bending moment based on the tensile strength of the cast iron. He was also able to calculate the distance which the neutral axis moved from its theoretical position through the center of gravity of the section.

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[*] Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

[†] *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 611.

[‡] *Loc. cit.*, pp. 673 and 679.

[§] *Loc. cit.*, p. 659.

[‖] "Shrinkage and Time Effects in Reinforced Concrete," by F. R. McMillan, Univ. of Minnesota, Studies in Engineering No. 3 (March, 1915).

[¶] "Elasticität und Festigkeit," Ninth Edition, p. 298; see, also, Figs. 18 and 19 of the writer's paper, "Stresses in Thick Arches of Dams," *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 508.

Bending tests on plain concrete beams have shown that the bending moment necessary to bring about failure is about twice that computed from the tensile strength of the concrete. This must be due to a shifting of the neutral axis, as was found for cast iron. The same phenomenon must make itself felt also when tension is not present; that is, when the resultant lies within the middle third of the section. This would mean that the structures have considerably greater strength than calculations with a constant modulus of elasticity would indicate, and that added strength has undoubtedly saved a number of poorly designed structures. Therefore, the fact that a structure which, according to theory, ought to fail and yet does not fail, does not mean that the theory is not to be relied on, but rather that the ordinary theory errs somewhat on the side of safety. This is a fortunate circumstance for engineers; it means that structures have, so to speak, a certain over-load capacity.

Professor Rathbun's tests should help to give confidence in theories based on the fundamental assumptions. He has performed a real service.

EDWARD GODFREY,* M. AM. SOC. C. E. (by letter).—Structural engineering in its proper functioning consists, not in working out a theoretical problem, but in building a structure that cannot be assailed from the standpoint of other theory, even if that theory appears to be of a lower grade and to savor of practical knowledge.

The writer is in agreement with George F. Swain, Past-President and Hon. M. Am. Soc. C. E.,† both in questioning the correctness of applying the elastic theory to concrete or reinforced concrete arches and in considering a skew arch as anything but an unsymmetrical arch subject to twisting stresses. Professor Swain gives some very sound reasons why the elastic theory is inappropriate for the solution of reinforced concrete arch problems. The extremely complex theoretical treatment of this subject should be eliminated and the "row-of-blocks" method, which is frankly an approximation and not camouflaged with meaningless and misleading mathematics, should be substituted.

Professor Swain states very aptly, "Nobody can ever know which method is the more accurate, but he should realize that more mathematics does not mean more accuracy". If the question were merely one of arriving at two nearly identical results by different paths, there would be no reason to object to any engineer's selection of the difficult mathematical path; but the objection which the writer urges against the elastic theory for a thing that can be proved not to be elastic is that a weaker structure undoubtedly is the result.

The one serious count against the elastic theory for reinforced concrete arches is that it assumes absolute rigidity in the abutments. This generally means that the design of the abutments is given no consideration whatever. The writer does not recall a single instance where a proponent of the elastic theory for arches considers the possibility of abutments not being absolutely, or even approximately, rigid. Furthermore, although replies to arguments against the elastic theory on the grounds of rotating, sinking, or shifting abutments state that this movement must be theoretically treated, no hint is given as to how the amount of rocking or shifting is to be determined.

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* Structural Engr., Pittsburgh, Pa.

† "Stresses, Graphical Statics, and Masonry."

Another basic assumption in the elastic theory is that the concrete is elastic. This is almost as serious an error as to assume fixed ends. To state that a thing is elastic from an engineering standpoint does not mean merely that it will return to its original shape after being stressed and the load removed. There are any number of heterogeneous combinations of springy materials, that could be made to fill this definition. To be elastic in the engineering, mathematical sense can only mean one thing, and that is that every part of the member so considered is not only elastic in the ordinary sense, but has the same modulus of elasticity. It is well known that concrete is far from fulfilling this condition. The modulus of elasticity of concrete is not constant. One mixture of given materials may have one value and another mixture of the same materials may have a value differing from the first by 50 to 100 per cent. The entire theoretical treatment of the arch is based on this value being constant, and no opinion can eliminate the bald fact that the whole treatment is so much waste effort if the modulus of elasticity is not even approximately constant. The elastic theory further assumes an original unstressed condition of the arch. This means that, if the arch could be submerged in a fluid of exactly its own specific gravity, it would assume precisely the shape that would fit its abutments. It is doubtful whether any one believes that shrinkage and expansion of concrete do not tend to distort any arch that can be built.

Professor Rathbun's tests proved the variability of the modulus of elasticity of plaster of Paris due to the use of varying quantities of water. Cement being, like plaster of Paris, a colloidal substance, will give a product varying in the same way. The argument that modern concrete is an exact product does not require a man to be on a field job more than one day to answer in the negative.

There is still another sense in which concrete is not elastic. Experiments have shown that, when subject to long-time tests, it takes a permanent set. This is a fact that need give little concern in an arch designed on the non-elastic basis with its liberal depth of arch ring. In fact this will tend to equalize the stresses. However, in the thin, reinforced concrete arch ring, designed on the elastic theory with part of the cross-section possibly in tension, a permanent set is apt to be serious.

Professor Rathbun's tests on skew arches seem to confirm his theory set forth in his earlier paper,* and they agree with that of Rankine for skew arches. Theoretically, such arches act as right arches; not showing, as many believe, a tendency for the load to take the short course to the abutments and overload the obtuse corners of such abutments. It is to be noted that in these tests extensive provisions were necessary to maintain fixed abutments. It is also to be noted that, when the abutments were allowed to shift only a little, the tendency to overload the obtuse corners was very manifest; that is, the reaction went to the obtuse corner. It is exactly this tendency that makes the skew arch, built as such, a menace.

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* "Analysis of the Stresses in the Ring of a Concrete Skew Arch," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 611.

In a laboratory on a small model it is possible to maintain rigid or fixed abutments. In a bridge, this is not a possibility. Tests on a full-sized arch bridge* (119 ft. span) have proved that rotation of the piers is perceptible although the piers are founded on rock. This recent light on the action of concrete arches and the instability of their abutments is of tremendous significance. If a pier founded on rock rotates, what would happen where the foundation is ordinary soil, and how is any complex theory, that makes for the absolute minimum thickness in an arch ring on the assumption of perfect fixedness of abutments, going to justify its existence?

Professor Rathbun's model test shows that a small endwise movement of a pier in the skew arch affects the distribution of the thrusts, throwing the main thrust into the obtuse corners of the abutments. In a bridge there will be a very heavy component of the thrust on the pier tending to move it in exactly that way. To take this thrust, there would have to be constructed special thrust-bearings, and these would have to bear against vertical surfaces of earth, which offer but uncertain resistance. Professor Rathbun recognizes the need of this provision in his Conclusion (2), but it is to be noted that in his tests it was taken care of by adjustment. No adjustment is possible in a concrete structure, and settlement is always one of the things to be expected. A settlement against a horizontal surface of earth in response to an active horizontal force is certain and unavoidable. Such settlement means that the reactions assumed in the design are not realized and that all the design calculations are in error.

The writer is in agreement with Professor Rathbun's† earlier ideas on the subject of skew arches, expressed in a letter in which, in commenting on the failure of a skew arch at Tacoma, Wash., he states:

> "By means of models it has been repeatedly shown that the thrust is greater at the obtuse corner [in plan] than at the acute. * * * Waddell, in 'De Pontibus', says, 'A skew bridge is a structure the building of which should always be avoided when it is practicable'. After a great deal of study on the theory of the skew arch the writer has come to the conclusion that Waddell was right, even if the remark was written before the days of reinforced concrete".

Clyde T. Morris, M. Am. Soc. C. E., has stated‡ that tests on a skew arch in a laboratory show that the greater part of the reaction is close to the obtuse corner. In some cases the reaction at the acute corner is negative. Hool§ states that,

> "Skew arches may be treated exactly as right arches, the span being taken parallel to the center line of the roadway and not at right angles to the springing lines of the arch".

It is this idea which Professor Rathbun's tests and comments foster, but which practical considerations show to be dangerous. The writer is of the opinion that, if a skew arch must be used, it should be done by constructing separate parallel ribs not bonded in such manner as to transmit shear from one to another.

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* *Engineering News-Record*, January 24, 1924, p. 159.

† *Loc. cit.*, April 12, 1923, p. 682.

‡ *Loc. cit.*, April 20, 1922, p. 638.

§ "Reinforced Concrete Construction," Vol. III, p. 43.

The Bendigo Arch, a skew structure that failed,* had especially massive foundations built on solid rock. The materials were excellent. However, the arch failed by crushing the abutments at the obtuse corners.

J. CHARLES RATHBUN,† M. AM. SOC. C. E. (by letter).—The experiment described in this paper was performed to test the validity of the assumptions made in developing the theory of the skew arch.‡ Subsequently, George E. Beggs, M. Am. Soc. C. E., conducted a far more elaborate set of experiments, using several arches of varying proportions. The writer compared the data from this set with the results deduced from his equations and found the comparison far more favorable than those described in the paper. Influence surfaces were reproduced to show the comparison for the entire set.

By making computations for varying values of F for an arch with dimensions otherwise the same, it has been found that little change in the results is produced if the variation is not greater than 20 per cent. In this case, the quantity, $\frac{\lambda}{F}$, was given values 20% greater and 20% smaller. If an extreme change is made the results are affected very materially. These computations were made to test the validity of the objections sometimes raised that the accuracy of the whole theory rests on that of the formula used for the value, F, which may not apply to a rectangle as slender as those used in arch design. The writer doubts very much if it is 20% in error.

A comparison of Mr. Godfrey's discussion with that of Mr. Jakobsen is very interesting, giving as it does the pro and con of scientific design *versus* "horse-sense" methods. The many catch problems that abound in engineering usually yield to a careful analysis, giving results that are different from the solution that at first appears obvious from the "horse-sense" method of attack. The skew arch is one of these problems. Although many engineers have advocated the idea that the obtuse corner carries the load, none of them has given a quantitative solution, so that designers can have a basis on which to compute stresses and earth pressures.

Mr. Godfrey emphasizes the fact that the abutments may not be rigid. The effect of abutment movement has been previously stated by the writer§ as follows:

> "The theory as developed implies rigid abutments. A positive movement of the abutment in the direction, Z, causes the line of force to shift toward the obtuse corner, that is, to tend to follow the shortest route to the abutment. A similar movement in the direction, X, however, will cause it to shift away from the obtuse corner. Rotation about a z-axis will cause a similar shifting either toward or away from the obtuse corner, depending on the direction of rotation."

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* *Engineering Record*, April 16, 1910.

† Prof., Civ. Eng., Antioch Coll., Yellow Springs, Ohio.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 611.

§ *Loc. cit.*, p. 675.

If the tangent of the angle of skew is somewhat less than unity the thrust in the direction, Z, is less than that in the direction, X. The tendency to move in the direction, X, is, therefore, greater than in the direction, Z, and the effect of the abutment movement would be opposite to that indicated by Mr. Godfrey. As the paragraph quoted was a very important one, it was verified experimentally.

Mr. Godfrey calls attention to the fact that the writer, before conducting experiments, and deducing the equations published in 1924, had fallen into the error still advocated as "horse sense". This is clearly a point in favor of the theory, as the experimenter was not trying to prove and publish a pet theory or, if he was, he was forced to conclude that he was in error after seeing the results of experiment and computations from the mathematical equations. It is hoped that the others who have committed themselves on this problem will admit that mathematical and experimental research have more weight than their first "snap" judgment.

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Paper No. 1726

FLOOD CONTROL ON THE RIVER PO IN ITALY*

By John R. Freeman,† Past-President, Am. Soc. C. E.

With Discussion by Messrs. C. McD. Townsend, Oren Reed, John P. Hogan, William B. Gregory, John R. Slattery, Arthur P. Davis, G. de Thierry, M. Giandotti, R. D. Goodrich, C. E. Grunsky, and John R. Freeman.

Synopsis

1.—This paper reports observations made by the writer on a tour in September, 1927.

2.—The Po is by far the most important river in Italy, drains one-third of its area, and flows through a broad valley rich in agriculture.

3.—The story published many times in the course of discussions on the control of the Mississippi River, that the bed of the River Po has been raised by deposit of sediment to elevations above that of the adjacent country, is found untrue.

It is true that there has been an increase in the elevation to which great floods have risen. Perhaps this is due to confining the waters within narrower limits and to the building up of deposits of sediment on the land between the main dikes and the river channel.

4.—Following the greatest flood of record, in 1926, works are now planned for raising the main dikes 3.3 ft. for many miles along the lower river.

The deposit of sediment brought down by the River Po, and precipitated where the fresh water meets salt water, has extended its delta 15 or 20 miles within the historic period, thus increasing the length of the river and adding about 160 acres of marsh land each year. A century ago the average was nearly 530 acres per year. This material comes chiefly from erosion of the slopes of the Apennines.

5.—The drainage area of the Po receives an average rainfall of 42 in. per year, varying from 120 in. on some of the elevated mountain slopes to only

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* Presented at the meeting of June 6, 1928.

† Cons. Hydr. Engr., Providence, R. I.

20 in. in the low broad central valley, which has an average width of 40 miles for the lower 175 miles.

6.—Great floods along the Po have occurred from time immemorial and the river and its torrential tributaries have long been diked for protection.

7.—Hydraulic science was born in the valley of the Po, hundreds of years ago, and its earliest treatises relate to the control of its floods and its torrents. Leonardo da Vinci, Galileo, Torricelli, Guillimini, Frisi, and many others of the most eminent scientists of one to four centuries ago, gave great attention to its problems.

8.—Breaks in the dikes in the great flood of 1917, and the desire for increased facilities for navigation led to a remarkably complete collection of hydrographic data. The number of rain gauges in this drainage area was increased to nearly 1 000, an average of 1 to each 30 sq. miles; river discharge gauging stations to 66, including 1 or more on each more important tributary; and 123 stations were established for frequent measurement of ground-water elevation within the broad valley.

These stations provide the foundation for what perhaps is the most intensive study of a problem in river hydraulics ever undertaken anywhere in the world.

9.—Vast areas of land that are slightly below sea level, including marshes and beds of old lagoons, have been reclaimed by dikes and pumping. The cause of their being below sea level doubtless is a slow earth tilt or subsidence of the land around the Adriatic that has been observed for several centuries.

10.—The main lessons from the Po which seem applicable to Mississippi and Missouri River problems, are:

(*a*) The use of double lines of dikes (or network of dikes) which presents a second line of defense when the main dike is breached.

(*b*) The maintenance of large brick storehouses at intervals of 3 to 5 miles, in which hundreds of thousands of sacks, such as grain sacks, are stored in readiness for emergencies of building sand-bag coffer-dams.

(*c*) The organization of the inhabitants so that thousands of men can be quickly mobilized in case of danger.

(*d*) A system of flood forecasts, and telegraphic warning of floods, with careful estimates of height that probably will be reached at important points all along the river.

(*e*) The prevention of "cross-over" sand-bars, at points of change of direction of curvature between the bends, by narrowing the river at these points. On the Mississippi these bars are chiefly caused by the extra width at points of contraflexure and consequent slower velocity during flood. By deposits due to this action the flood obstructs its own discharge and rises to greater heights.

(*f*) The effort to minimize cost of dredging by training the river into a single, relatively narrow, channel.

(*g*) The minimizing of expenditure for revetment where funds are scarce by trying to retain the river in the channel in which it has for a time appeared to be content, without attempting to shorten its course by short cuts across the bends.

(*h*) The present program of channel improvement on the Po is tentative and subject to revision. Some of the most eminent hydraulicians in Italy believe that a hydraulic laboratory would aid in improving methods. Plans for a research laboratory at Stra, larger than those developed for purposes of instruction at the near-by engineering schools, have been outlined; but funds have not been available for its construction.

(*i*) The work of improvement is in charge of civil (not military) engineers; with an *ex*-college professor in charge of the wonderfully elaborate preliminary hydrographic work. The whole organization reports to the Department of Public Works. Some of the most eminent engineers of Italy, consultants, construction specialists, and professors in engineering colleges, were called upon to formulate the procedure.

INTRODUCTION

Because of statements here and there in engineering literature and encyclopedias, important if true (found to be mostly untrue), that the bed of the River Po had been raised above the elevation of the adjacent lands because of its enclosure by dikes for flood control, and the consequent deposit within the river bed of sediment which otherwise would have been spread out over broad areas of flooded land, the writer has long been trying to learn more about flood conditions, dikes, sediments, and elevation of river bed along this remarkable river.

The Po (Fig. 1) is the largest river in Italy. It drains much of the southern slopes of the Alps and most of the northern slopes of the Apennines, and with its tributaries, flowing through the broad fertile plain of Lombardy—a valley filled to a nearly level floor by its sediments—has for a thousand years presented outstanding problems in river control. It was in this region that hydraulic science had its birth. It was on one of its tributaries near Milan that Leonardo da Vinci, genius of widest range in all history, practiced canal engineering and constructed the earliest chamber lock for navigation (Porte Vinciane).

It has been stated that the raising of the river bed by the increase in the deposit of sediment from year to year, has compelled the height of the dikes to be continuously increased in order to restrain the higher floods from spreading over one of the world's most fertile and intensively cultivated delta plains. Flood heights certainly have increased, but the best data available show that meanwhile the bed of the main channel has not been raised. The cause may be the gradual building up, by deposits, of the flood-plain between the main dikes, and the greater confinement of the water by subsidiary dikes. So far as this condition exists, it pertains to the steeper tributaries and to the adjacent Rivers Reno and Adige, and is limited to those reaches where the steeper slope of valleys between the hills changes to the lesser slope across the floor of the main valley.

Assuming the flood level to be about the same as the altitudes of the valley cities reported in the Encyclopædia Britannica, the flood slopes would be:

Piacenza to Cremona, 62-ft. fall in 29 miles, averaging about 2.1 ft. per mile.
Cremona to Ferrara, 125-ft. fall in 110 miles, averaging about 1.1 ft. per mile.
Ferrara to the sea, 30 ft. fall in 50 miles, averaging about 0.6 ft. per mile.

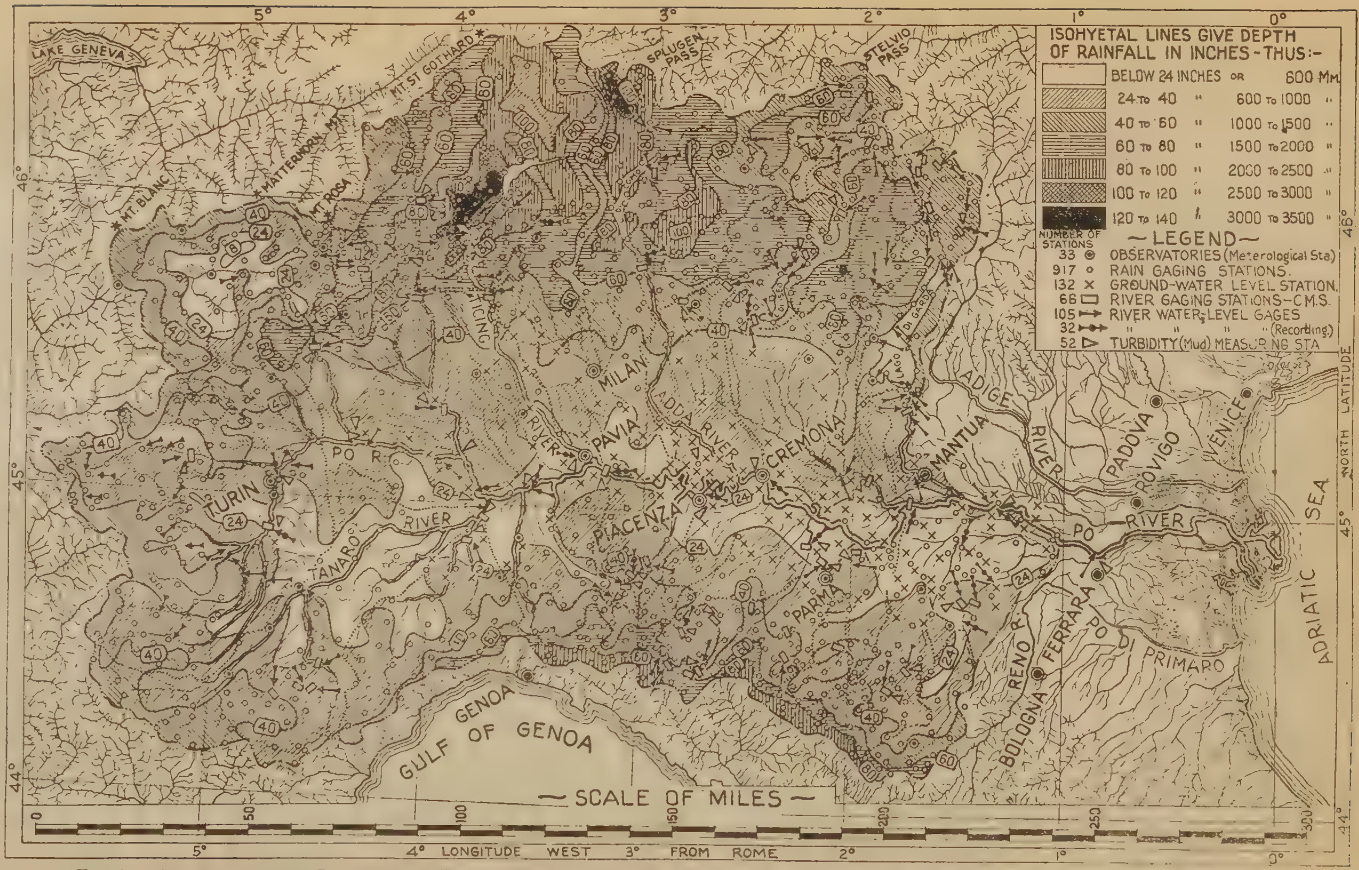
ISOHYETAL LINES GIVE DEPTH OF RAINFALL IN INCHES - THUS:-
BELOW 24 INCHES OR 600 MM
24 TO 40 " 600 TO 1000 "
40 TO 60 " 1000 TO 1500 "
60 TO 80 " 1500 TO 2000 "
80 TO 100 " 2000 TO 2500 "
100 TO 120 " 2500 TO 3000 "
120 TO 140 " 3000 TO 3500 "
—LEGEND—
NUMBER OF STATIONS
33 ◉ OBSERVATORIES (Meterological Sta)
917 ○ RAIN GAGING STATIONS.
132 × GROUND-WATER LEVEL STATION.
66 ▭ RIVER GAGING STATIONS-C.M.S.
105 RIVER WATER-LEVEL GAGES
32 " " " (Recording)
52 ▷ TURBIDITY (Mud) MEASURING STA
LAKE GENEVA
MT. BLANC
MATTERHORN MT.
MT. ROSA
MT. ST. GOTHARD
SPLUGEN PASS
STELVIO PASS
TICINO
MILAN
ADDA RIVER
PAVIA
RIVER
CREMONA
MANTUA
LAGO DI GARDA
ADIGE RIVER
PADOVA
ROVIGO
VENICE
TURIN
PO R.
TANARO RIVER
PIACENZA
PARMA
PO RIVER
RENO R.
FERRARA
PO DI PRIMARO
BOLOGNA
GENOA
GULF OF GENOA
ADRIATIC SEA
—SCALE OF MILES—
0 50 100 150 200 250 300
5° 4° LONGITUDE WEST 3° FROM ROME 2° 1° 0°
46° 45° NORTH LATITUDE 44°

FIG. 1.—DISTRIBUTION OF RAINFALL, IN INCHES DEPTH, FOR HYDROLOGIC YEAR, 1923 24, ON THE BASIN OF THE RIVER PO, ITALY.

Nearly all rivers decrease in declivity down stream, presenting a curved profile. Obviously, within the lower delta the slope must vary greatly with the river stage from flood to drought, because at 23 miles from the sea the flood surface is 20 ft. above its low-water elevation. This would give a flood slope of nearly 1 ft. per mile where the low-water slope is only about 0.10 ft. per mile.

An outstanding fact that may have had more or less effect on the downstream 200 miles of the main river, in raising both flood level and river bed, is that about 15 or 20 miles have been added to the length of the river since Roman times by the projection of its delta into the Adriatic through the deposit of sediment at its outlet, and that additional height or fall is required to carry the water over this added distance.

In the many controversies of 20 to 40 years ago, about adopting the levee system for the Mississippi, it was often stated (and also disputed) that confinement by dikes had raised the bed of the Po in Italy and of the Yellow River in China. The writer visited the Yellow River and had lines of levels run across its course at many places in the 240 miles between the crossing of the Peking-Hankow Railroad, and that of the Tientsin-Pukow Railroad, which proved those reports about the Yellow River to be mostly untrue, although not entirely so.*

CONDITIONS PRESENTED ON THE PO

The total drainage area of the Po (Fig. 1) is about 28 000 sq. miles, or nearly one-third the area of all Italy. Its breadth from north to south averages nearly 100 miles. Its length from west to east in a straight line is only about 300 miles, although about 425 miles along the curving course of the river. Nearly one-fourth this nominal drainage area, or about 7 000 sq. miles, comprising the nearly level floor of the valley, which for its lower 1 750 miles averages 40 miles in width, does not drain directly into the river because of being enclosed by dikes which while keeping out the flood water that comes from up stream, hold the rain that falls on the surface inside the dikes. Much of the rain water soaks into the porous ground, some slowly escapes through drainage channels, and a large part goes skyward in evaporation.

The average annual rainfall for the entire area is 42 in., which is about the same as on New England rivers. On some of the high mountain slopes the total annual precipitation averages more than 100 in., while it averages only 24 in. on the broad valley floor. Data for the great flood of 1927 had not been published at the time of the writer's visit.

The three successive storms that caused the previous great flood of 1917, namely, June 3 to 10, of 1.97 in. depth of rainfall, June 19, of 2.68 in., June 28 to 31, of 5.36 in. (Fig. 2), an average total of 10.01 in. of rainfall on about 20 000 sq. miles within 28 days, would cause trouble almost anywhere.

The maximum run-off of about 300 000 cu. ft. per sec. in 1917 was about 11 cu. ft. per sec. per sq. mile if reckoned on the entire area; but if reckoned

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* "Flood Problems in China," by John R. Freeman, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1422.

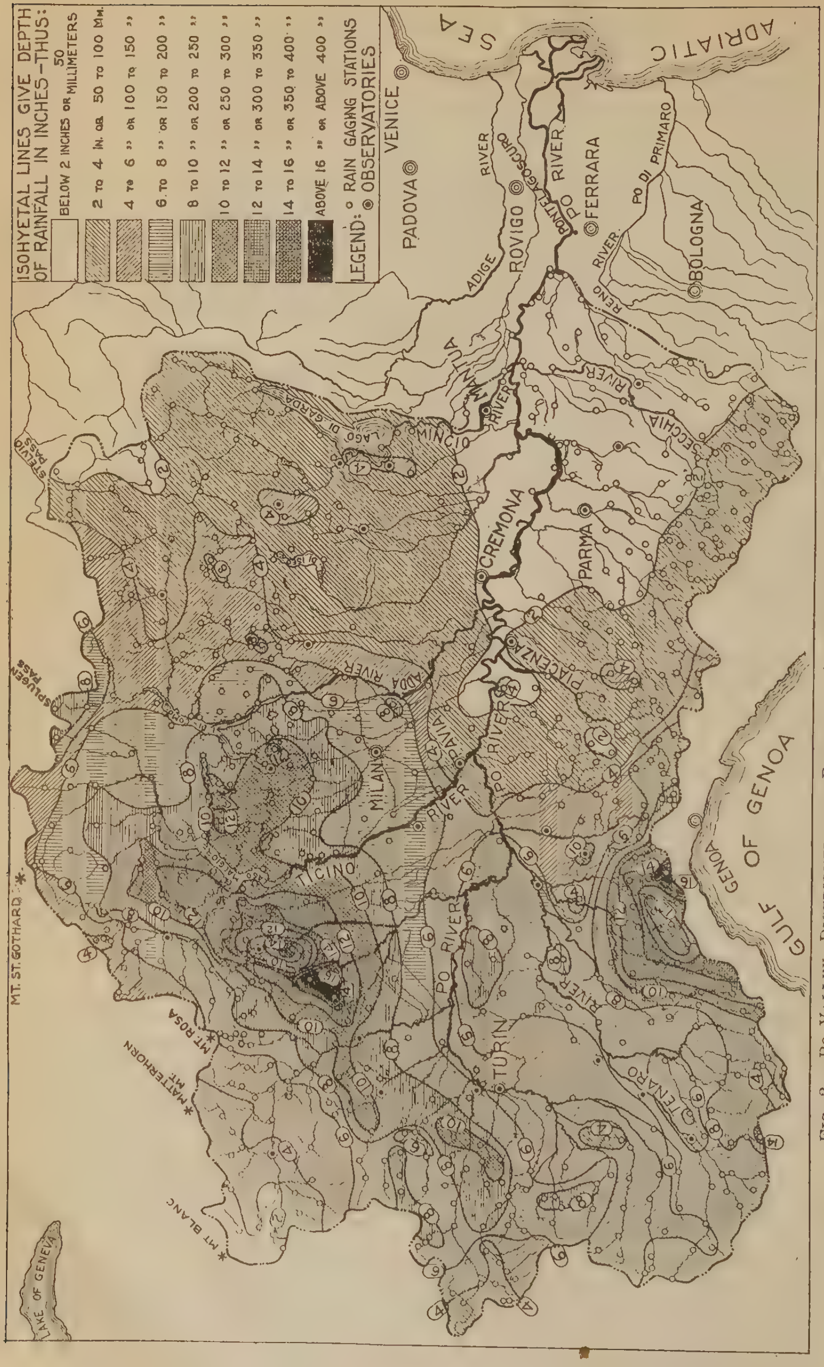
ISOHYETAL LINES GIVE DEPTH OF RAINFALL IN INCHES–THUS: 50
BELOW 2 INCHES OR MILLIMETERS
2 TO 4 IN. OR 50 TO 100 MM.
4 TO 6 " OR 100 TO 150 "
6 TO 8 " OR 150 TO 200 "
8 TO 10 " OR 200 TO 250 "
10 TO 12 " OR 250 TO 300 "
12 TO 14 " OR 300 TO 350 "
14 TO 16 " OR 350 TO 400 "
ABOVE 16 " OR ABOVE 400 "
LEGEND: ○ RAIN GAGING STATIONS
◎ OBSERVATORIES
ADRIATIC SEA
VENICE
PADOVA
ADIGE RIVER
ROVIGO
CANAL BIANCO
PO RIVER
PONTELAGOSCURO
FERRARA
PO DI PRIMARO
RENO RIVER
BOLOGNA
MANTUA
MINCIO RIVER
LAGO DI GARDA
SECCHIA RIVER
CREMONA
PARMA
PIACENZA
ADDA RIVER
STELVIO PASS
SPLUEGEN PASS
MILAN
PAVIA
PO RIVER
TICINO RIVER
MT. ST. GOTHARD
MT. ROSA
MT. MATTERHORN
MT. BLANC
LAKE OF GENEVA
TURIN
PO RIVER
TENARO RIVER
GENOA
GULF OF GENOA

Fig. 2.—Po Valley, Distribution of Rainfall, Averaging 5.36 Inches from May 28 to 31, 1917.

on the effective area, it was about 15 cu. ft. per sec. per sq. mile. This is equivalent to a depth of run-off of only 0.6 in. per 24 hours. This seems small for the record run-off of a century when one considers the depth of rainfall, the lay of the land, and the scant forest cover. Explanation may perhaps be found in an uncommonly absorptive soil, the surface of which at times of rain is never sealed by frost over any large proportion of the whole surface.

The northern edge of this water-shed is formed by the high Alps, rising from 1 mile to more than 2 miles above sea level; and its southern edge by the Apennines, some of the peaks of which are nearly a mile high.

The data given in Fig. 3 show a much more uniform distribution of rainfall and of run-off month by month than on most American drainage areas. The annual *Bulletin* of the Hydrographic Survey of the Po for 1923-24, published by the Department of Public Works, presents an admirably complete record, analyzed with great thoroughness. It forms a volume of two hundred and fifty, 10 by 14-in. pages, with large scale maps of the isohyetals and many diagrams of discharge.

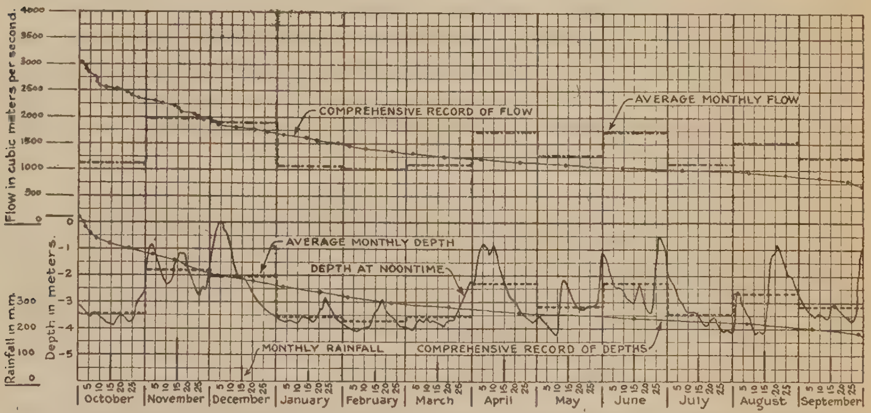

FIG. 3.—RELATION OF MONTHLY RAINFALL ON TOTAL WATER-SHED TO HEIGHT OF RIVER PO AT PONTELAGOSCURO, FOR HYDROLOGIC YEAR, 1923–24.

In brief, the deductions from the records of the 883 rain gauges in service for that year gave an average precipitation weighted according to areas adjacent to each gauge by mapping, of 1 085 mm., or 42.7 in., for 1923-24, and 982 mm., or 36.7 in., for the year next previous. On 68.5% of the drainage area it exceeded 1 200 mm., or 47 in., in 1923-24. In the six months—October to March—there was a total average precipitation of 496 mm., or 19.5 in.

The continuous gaugings of discharge at Pontelagoscuro indicate a total quantity of water equivalent to a sheet, 630 mm. (equal to 25 in.) in thickness over the entire drainage area, which, since the average precipitation was 1 085 mm., gives a run-off of 58% of the precipitation. For 1922-23, this ratio was 57%; for 1921-22, 52%; and for 1920-21, 68 per cent.

The turbidity measurements showed that during the year about 6 170 000 metric tons computed as dry silt passed the gauging station at Pontelagoscuro, which corresponds to an average erosion of 113 tons from each square kilometer of the drainage area, or equivalent to about 0.0029 in. in depth per year, or ⅕ in. per century, from the entire drainage area; but, as stated elsewhere, most of this comes from the steep slopes of the Emilian Apennines.

The lay of the land in this great valley of the River Po presents conditions on all sides which one would expect to aggravate sudden floods—high mountains, moist air swept northward from the sea, steep declivities, short swift mountain currents—all bringing the rainfall rapidly from north, south, and west, into a broad and nearly level valley floor. This floor is roughly 150 miles long by 50 miles broad, and has been formed during hundreds of thousands of years by the sand and mud eroded from the mountain slopes and brought in by the currents, filling up a broad valley that doubtless once was deep and of V-shaped cross-section, but which now presents a relatively small declivity for drainage toward the sea. Within this present broad and nearly level plain, the main river and its many tributary rivers are confined by dikes from overflowing the highly cultivated land.

That part of the delta formed within the past 2 000 years, comprising about 750 sq. miles, or nearly 3% of the entire drainage area, lies down stream from the ancient City of Ferrara and covers what once was the apex of the Gulf of Comacchio. Delta extension has been going on during the past two or three centuries at the rate of nearly ½ sq. mile per year. Within this lower delta are vast areas of low land, part of it a few feet below sea level, that have been reclaimed from shallow lagoons and marshes by dikes and pumping, mostly within the past fifty years.

Prospects for Useful Information—Historic Backgrounds

From a study of the literature it seemed highly probable that, by a brief personal tour of observation along this river, facts could be learned that would at least be of interest in the discussion, and, perhaps, in some small degree aid in the solution of the flood problem of the Mississippi and its tributaries.

The science of hydraulics had its birth in Northern Italy and was in part developed 400 to 200 years ago by some of the greatest scientists of any age: Leonardo da Vinci, Galileo, Torricelli, Castelli, Viviani, Guglielmini, Grandi, Manfredi, Frisi, and others in their efforts to restrain these torrents, and safeguard the inhabitants along the valley and the delta of the Po.*

No region in the world was so rich in hydraulic literature up to 100 years ago, as this valley of the River Po. Some of the most ancient and famous universities and engineering schools of Europe, including those of Milan, Turin, Padua, Pavia, and Bologna, are located in and near this valley. Bologna claims the oldest university in Europe and Padua has one nearly as old. Pavia has been a center of study for more than 1 000 years. Columbus studied at its university, and, here, Volta made his first experiments. Foremost among "the old masters" of hydraulics were Michelotti, a professor at Turin; Frisi, a professor at Milan; and Poleni, at Padua. Guglielmini, who wrote the foremost treatise of his time, in 1697, on "The Nature of Rivers", was head of the Department of Mathematics in the Royal Academy of Science at Bologna and from 1700 to 1710 a professor at the University of

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* The Pontine marshes, which present problems of drainage that have engaged the attention of several of the most eminent engineers of Europe, are not connected with the River Po. They are located near the west coast, about 50 miles south from Rome.

Padua.* The 11 remarkable volumes comprising, in their 5 500 pages, no less than 63 separate treatises by Italian hydraulicians of the preceding centuries, on the motion of water and the control of rivers, collected and republished more than 100 years ago under the title of the "Nuova Raccolta", was printed at Parma and Bologna within this region and related largely to the River Po and its tributaries.† Frisi's famous treatise on rivers and torrents, the world's outstanding book on these matters a century and a half ago, twice translated for the benefit of English engineers in India, deals mainly with the problems of this valley of the Po. History shows that from 1 000 to 200 years ago the Po was restless, meandering, and occasionally changed its bed. Frisi states that many centuries ago the river divided into several branches between Piacenza and Parma and converted a considerable part of Lombardy into a swamp which has since been circumscribed by dikes, while the river has been confined to a single channel. Ancient treatises presenting conditions of 900 to 1 000 years ago describe vast swamps that have since been transformed into agricultural land along the lower 170 miles of its course.

About 800 years ago the down-stream portion of the Po turned to the north, forming the Venetian branch, which gradually increased and for a time absorbed the water that formerly flowed into the southerly branches. About 325 years ago, after the Reno, which flows northward past Bologna (Fig. 2), had become so obstructed by sediment that it ceased to flow into the Grand Po, its waters spread over the shallow "valleys", or lagoons of San Martino, which thereby became filled and fertilized by sediment. Farther down stream these waters from the Reno, unconfined by dikes and without a permanent river bed, for a time ruined some of the most fertile land in Italy, formed vast swamps, and reached the sea slowly by the Po di Primaro. The Grand Po had formerly received both the rivers, Panaro and Reno, on its way to Ferrara, and, some miles below that city, it formerly divided into two branches, the Primaro and the Volano.

Frisi states further that the Reno and the Po are the two rivers that more than all others had engaged the attention of the Italian scientists, although the Arno and the Tiber also gave them much concern. The foremost mathematicians and scientists of Italy were called upon to find a remedy for the swamps and shifting waters of the Reno and the torrents lying easterly from it. They proposed to force the Reno to re-enter the Grand Po. Political obstacles, divided councils, accidents, and outbreaks long delayed the work, but finally the Reno was separated from the Grand Po and diverted into the old channel of the Primaro (Fig. 2).

Parts of the present channel of the Reno-Primaro, as outlined in modern maps, give evidence of artificial origin by their straight course. This channel was constructed within the southern edge of the great valley, and now gathers in various torrents—the Reno, Savena, Idice, Sillano, etc., flowing northward from the slopes of the Apennines, after they have dropped their loads of gravel in their separate delta cones subsequent to entering upon the more gentle slope of the valley floor. This information and much more that can be found in Frisi's treatise and in the eleven volumes of the "Raccolta", make it plain that this remarkable river and valley have received through the past four centuries the attention of the foremost engineers and physicists of the times.

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* Bologna is in the same valley on the River Reno that discharges into the Po di Primaro, which was the main channel of the River Po in the remote past.

† A full set of these remarkable books is in the Engineering Societies Library, New York, N. Y., the gift of Professor Fantoli, head of the Engineering College of Milan.

Two hundred years ago the Reno, near Bologna, formerly a tributary to the Po, seems to have given more trouble than the main river, and, in 1760, Frisi was called in for further study. He has much to say about the raising of river beds, but it pertains to rivers of much steeper slope than the main Po, and mostly applies to the Arno, which flows past Florence and Pisa. His views still make interesting reading, and indicate plainly that the instances of elevation of river bed were not on the main River Po, nor caused by confining its waters between dikes and preventing the spread of sediment over the land. The troublesome elevations of river bed occurred on the tributaries near where they left the steep slopes of the narrow valleys and entered upon the much more gentle slope across the broad valley floor, where the lessened velocity caused them to drop their burden of gravel.

Frisi argued that any new channel for carrying the waters of the Reno and the five torrents easterly therefrom to the sea, should have its pathway laid out well over into the valley floor of the Po, so as to receive the many torrents and small rivers that drain northward at a point beyond where they deposit their load of gravel brought from the hills, or beyond their gravel delta cones; and he appears to have felt certain that after a river had dropped its burden of transported gravel and carried only fine sand, it would not build up its bed. The main River Po drops its burden of gravel and coarse sand up stream from Cremona, or 160 miles above its mouth.

Frisi emphasizes gravel as a cause of restlessness in rivers, and states that the main Po drops its own load of gravel where it enters upon its relatively gentle slope at the head of the main valley, and that it receives little or no gravel from its many tributaries. Frisi's views as to the influence of gravel may have important application to the cross-over bars of the Mississippi, which are among its most troublesome features.

His books also relate that spillways for ameliorating flood conditions were actively considered along these Italian rivers more than 150 years ago, and, in general, were found to be inexpedient or unsuccessful; but these were relatively small affairs. After reading Frisi, one has a strong desire to extend his historical studies and make inspections all along the Reno from Bologna to the sea, but the writer had not time for this.

A Tour of Inspection

After preliminary talks with two or three of Italy's foremost hydraulic engineers, Professor Gaudenzio Fantoli, at Milan; Luigi Luiggi, Hon. M. Am. Soc. C. E., and Mr. Lorenzo Allievi, at Rome, who provided kindly introductions to the chief engineers of river control in the Department of Public Works, the writer spent four or five busy days inspecting the delta, the dikes, and the channel of the Po and the Adige Rivers. He traveled by automobile back and forth about 200 miles, largely on well-kept roads on tops of dikes, crossing and re-crossing the rivers repeatedly by bridge and ferry, and thus seeing much of them, their dikes, and the valley floor. This inspection included much of the way from Piacenza, past Cremona, Parma, Mantua, Padua, Ferrara, and Rovigo to the Adriatic, thence back to Ferrara, and across the valley floor to Bologna. The centuries of intensive cultivation and sub-division have given far more numerous cross-roads than there are along the Missouri, and nearly every road traveled was on top of an embankment that would form a secondary dike in case of need.

Many of the main dikes were said to be hundreds of years old, but had been repaired, raised, and thickened from time to time. Their permanence of location seems to indicate a less unruly river than the Missouri or Mississippi. Nevertheless, one look at the map of roads embanked on top of secondary dikes and at their curves which evidently follow old river bends, shows that this river has in past years sometimes meandered far outside its present pathway. (See Figs. 4 and 5.)

On most of the trip the writer was accompanied by one or another of the engineers in charge of the new work. The journey was extended by boat to the end of the principal delta mouth, where works are in progress for creating and maintaining a broad channel about 1 000 ft. wide, through the bar, suitable for barges of 600 tons capacity.

The tour was full of interest, the roads were excellent, and the farming was intensive, the fertile ground being burdened with crops, rice, maize, hemp, and forage. Economy in use of the land was shown by the long rows of trees along the cross-roads, planted to give forage for silk worms, and by occasional recent plantings on the foreshore, outside the dikes, of long parallel lines of poplars, about 12 ft. apart, for future wood pulp. There are now many broad strips of land of small utility lying between the main dikes and the river, which apparently it is the intention to reclaim for more intensive cultivation after the flood channel has been further stabilized and narrowed by works now in progress.

Everywhere the greatest courtesy was extended, many maps and reports showing features of special interest were furnished, and one was made to feel that the Italian engineer has a most kindly feeling toward his American co-worker. Space permits only a few of the observed features of special interest to be given.

WHAT WAS SEEN AND HEARD ON THE TOUR

All things considered, the writer knows of no more ambitious, courageous, and carefully planned project of river regulation now going on anywhere in the world than that in progress upon the Po, nor of any which rests on a more painstaking preliminary scientific study of the river itself.

For a number of years, a new hydrographic survey has been going on in the Valley of the Po, largely under the direction of Professor Mario Giandotti, following the recommendations of a Royal Commission of Engineers chosen from the most eminent in Italy, on which were Luigi Luiggi, Gaudenzio

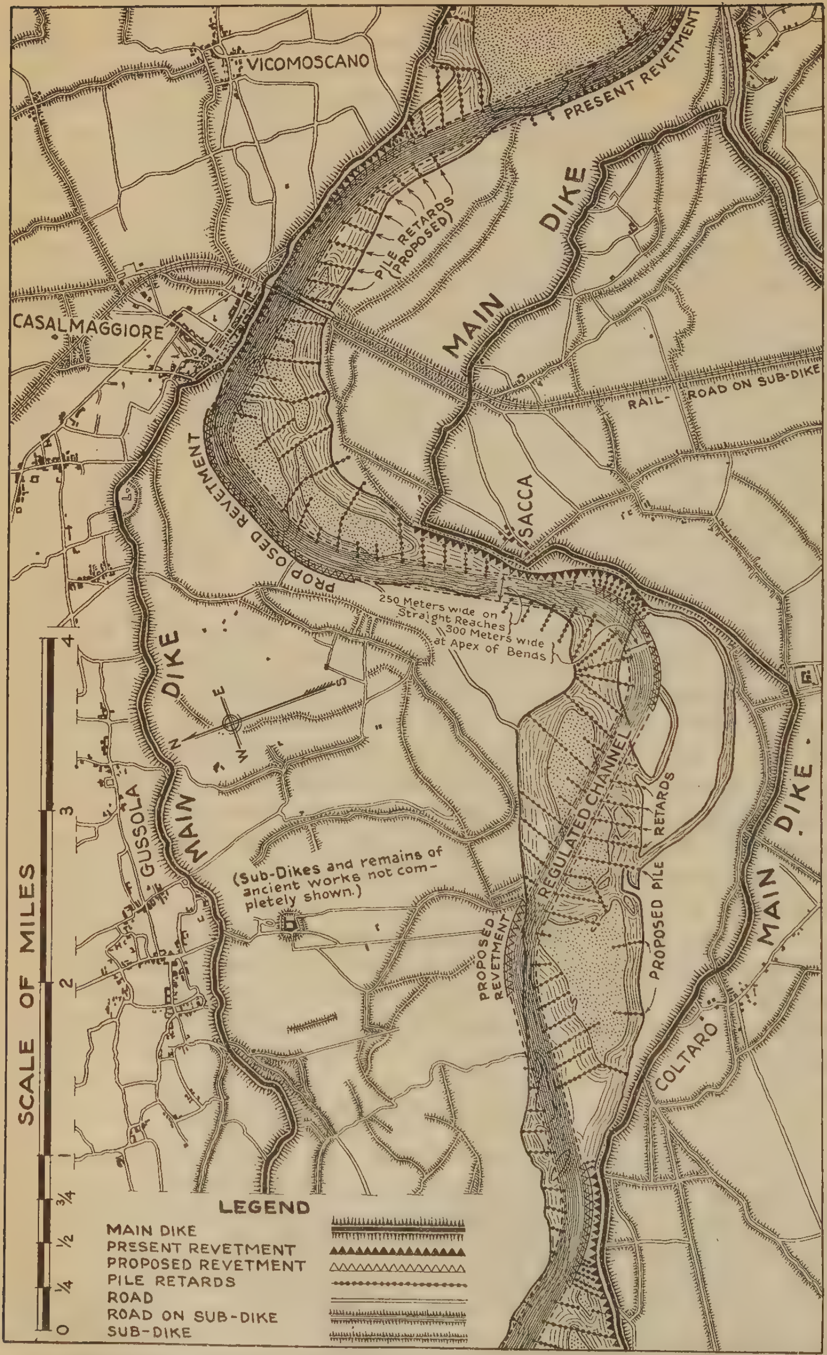
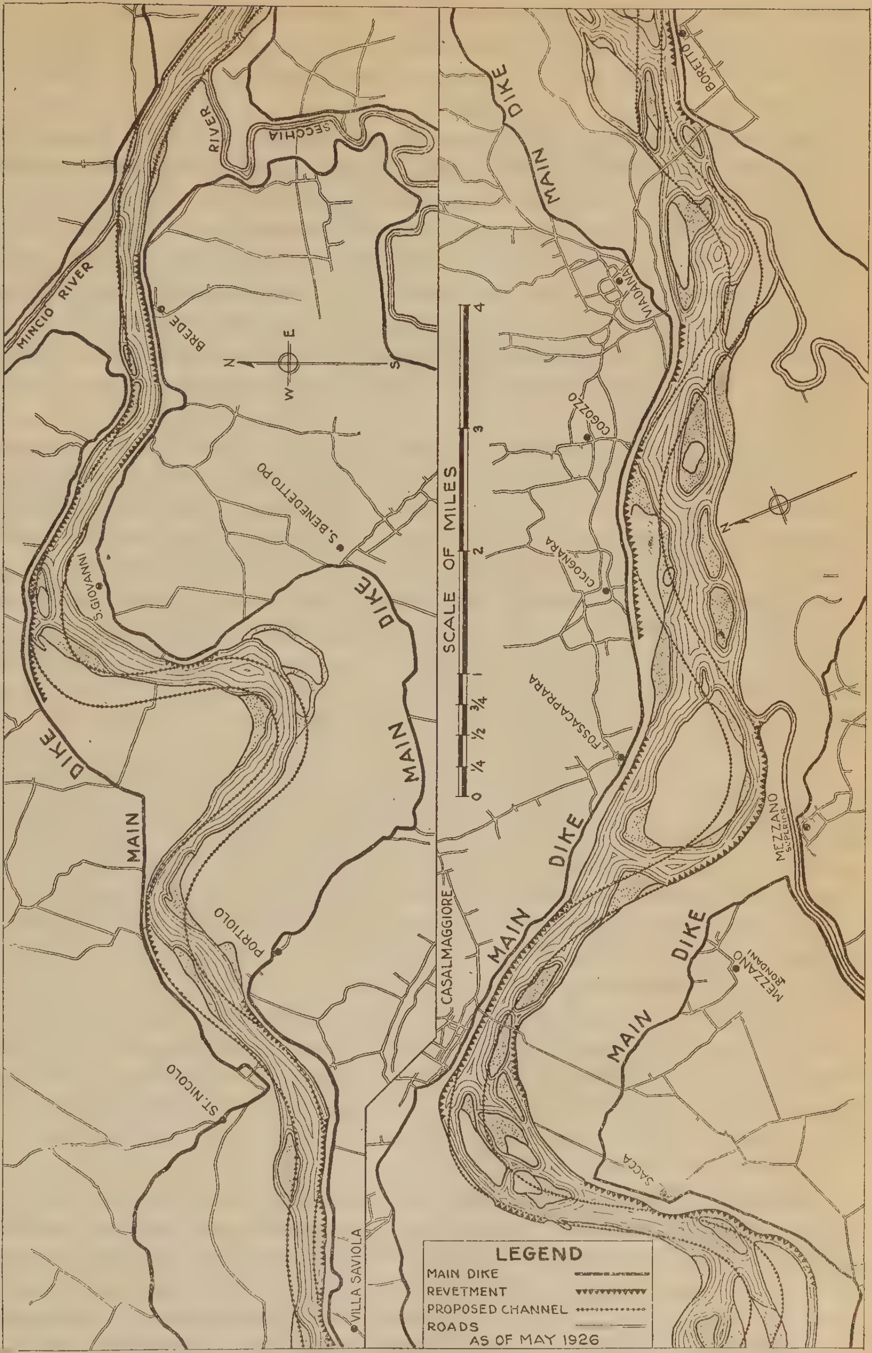

VICOMOSCANO
PRESENT REVETMENT
PILE RETARDS (PROPOSED)
MAIN DIKE
CASALMAGGIORE
RAIL-ROAD ON SUB-DIKE
PROPOSED REVETMENT
SACCA
250 Meters wide on Straight Reaches
300 Meters wide at Apex of Bends
DIKE
MAIN
GUSSOLA
N
E
S
W
(Sub-Dikes and remains of ancient works not completely shown.)
REGULATED CHANNEL
RETARDS
PROPOSED PILE
PROPOSED REVETMENT
MAIN DIKE
COLTARO
SCALE OF MILES
0
¼
½
¾
1
2
3
4
LEGEND
MAIN DIKE
PRESENT REVETMENT
PROPOSED REVETMENT
PILE RETARDS
ROAD
ROAD ON SUB-DIKE
SUB-DIKE

FIG. 4.—PRESENT AND PROPOSED CHANNEL AND REGULATING WORKS NEAR CASALMAGGIORE IN 1919.


SECCHIA
RIVER
MINCIO RIVER
BREDE
N
E
S
W
S.BENEDETTO PO
S.GIOVANNI
DIKE
MAIN
DIKE
MAIN
PORTIOLO
ST.NICOLO
VILLA SAVIOLA
DIKE
MAIN
BORETTO
VIADANA
COGOZZO
CICOGNARA
FOSSACAPRARA
SCALE OF MILES
0
1/4
1/2
3/4
1
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4
N
MAIN
DIKE
MEZZANO SUPERIORE
CASALMAGGIORE
MAIN
DIKE
MEZZANO RONDANI
SACCA
LEGEND
MAIN DIKE
REVETMENT
PROPOSED CHANNEL
ROADS
AS OF MAY 1926

FIG. 5.—SECTIONS OF RIVER PO, SHOWING TYPICAL REGULATING WORKS AND PROPOSED CHANNEL.

Fantoli, Mario Giandotti, Francesco Mauro, and others. Although many of the engineers now engaged on the work were in active service in the World War, they are not primarily Army engineers. All now report to the Ministry of Public Works.

Many large volumes of reports have been published relative to storm paths, isobars, rainfall, run-off, irrigation, effect of forests, and the character of substrata as shown by deep wells. These include a remarkably complete analysis of the great flood of 1917 and of the rainfall that produced it, comprised in 186 quarto pages. Their admirable isohyetal maps (see Fig. 1) show rain-gauge stations scattered over the thirty-two tributary sub-divisions of the catchment area, with a profusion that the writer has never seen equalled elsewhere. The reason for this lies in the rugged mountain topography of more than half the drainage area and the wide differences in precipitation at stations a few miles apart.

A count (Fig. 1) reveals 917 rain gauges, within this 27 500 sq. miles, or 1 to each 30 sq. miles. Sixty-six river discharge gauging stations are maintained, one on each important tributary. The river's height is regularly measured at 137 stations, of which 32 are autographic. There are 33 meteorologic observatories within this drainage. Determinations of the percentage of solid matter in suspension at various stages, particularly in floods, are being made at 52 different places, and samples of deposits of sediment from many localities are given mechanical analysis. The elevation of the ground-water is recorded at 123 stations, widely scattered over the valley floor. These various data are being studied and tabulated in excellent form in a series of printed reports.

This thoroughgoing investigation under the Italian Department of Public Works appears to be far better than anything of the kind that has been done by the U. S. Army engineers, or others, along the Mississippi, where scientific data for the more intelligent planning of flood control have long been needed. The only research of similar quality ever made in the United States, or elsewhere, as far as the writer has been able to find, is that done under the supervision of Arthur E. Morgan, M. Am. Soc. C. E., on the Miami River, in Ohio, following the great flood of 1913.

Relation curves of the rate of run-off to the river stage, also to the rate and total amount of precipitation, are being established for each important tributary and for the main river, in such form that immediately following any noteworthy rainfall a forecast of river height for the following day can be telegraphed to the cities along the stream. Supervisors are thus kept on the alert for any flood of importance.

This study of floods does not stop with reports and treatises. Brick storehouses are built at intervals of 3 or 4 miles along the river dikes, in which are stored tens of thousands of bags similar to grain sacks, all in readiness to be taken to any danger-point, filled with sand with the utmost celerity, and used for "hooping" sand-boils, for weighting and stabilizing a bank which threatens to slip because of saturation or pressure, or for building sand-bag coffer-dams on top of the dikes. The peasant farmers of the broad countryside are organized in readiness for extreme floods, each man knowing his appointed place along the dikes, so that literally thousands of men can be mobilized at a few hours' notice for defence at any point.

Proof of the efficiency of this organized foresight was given in the flood of 1926, which was the greatest ever recorded in the history of the Lower Po. In May, 1926, following extremely heavy and widespread rainfall, the river rose to a height averaging about 1.3 ft. higher than the top of the main dikes for many miles along the Lower Po, but was prevented from overflowing them. About 30 000 men are said to have been mobilized and, in addition to other work, built a continuous line of sand-bag coffer-dams, from 2 to 3 ft. high, and more than 30 miles in total length, so promptly and quickly that there was no break in the dikes down stream from Casalmaggiore, or along the lower 130 miles of the river. This was a wonderful performance, considering that the period of warning was remarkably short, and that the tributaries entering the flat valley floor are mostly short, steep, mountain torrents. The distance in an air line from the farthest corner of the water-shed up stream from Casalmaggiore is only about 190 miles.

The middle of the Po Valley, above Casalmaggiore, did not fare so well in the flood of May, 1926, or in that of June, 1917. Here and there were breaks and overtopping of dikes such that a large area of farm land was put under water, and city streets were flooded 2 to 5 ft. in depth. Photographs of the flooded districts bear striking resemblance to scenes the writer has witnessed along the Mississippi. There were the same sand-bag dams on top of dikes, the same weighting of insecure dike slopes with sand-bags, the same hooping of sand-boils, and a similar gathering of women, children, and household goods on the tops of the dikes that protruded from the flood. It is a strange coincidence that the 1926 flood on the Po and the 1927 flood on the Mississippi should have each attained the greatest height ever recorded.

Within the upper part of the delta plain of the Po, up stream from Casalmaggiore, more fortunately than along the Mississippi, the vast network of interior dikes, one of which forms the base of almost every road and cross-road, appears to have restrained the flood that penetrated the main dikes. Judging from their shape, some of these old dikes mark the outline of river beds of centuries ago.

Works are now in progress (1928) for many miles along the Po, for raising the main dikes 1 m., or to 3.3 ft., above their present elevation.

DISTRIBUTION OF RAINFALL

The two maps, Figs. 1 and 2, are selected from among many contained in recent reports, as showing the remarkably uneven distribution of the precipitation caused by the contrasts in elevation and exposure to vapor-laden winds. The first (Fig. 1) shows the distribution of the average total precipitation for the entire year; and Fig. 2 shows that for the 2½ days, May 28 to 31, which caused the great flood of 1917, one of the most severe recorded within the past century.

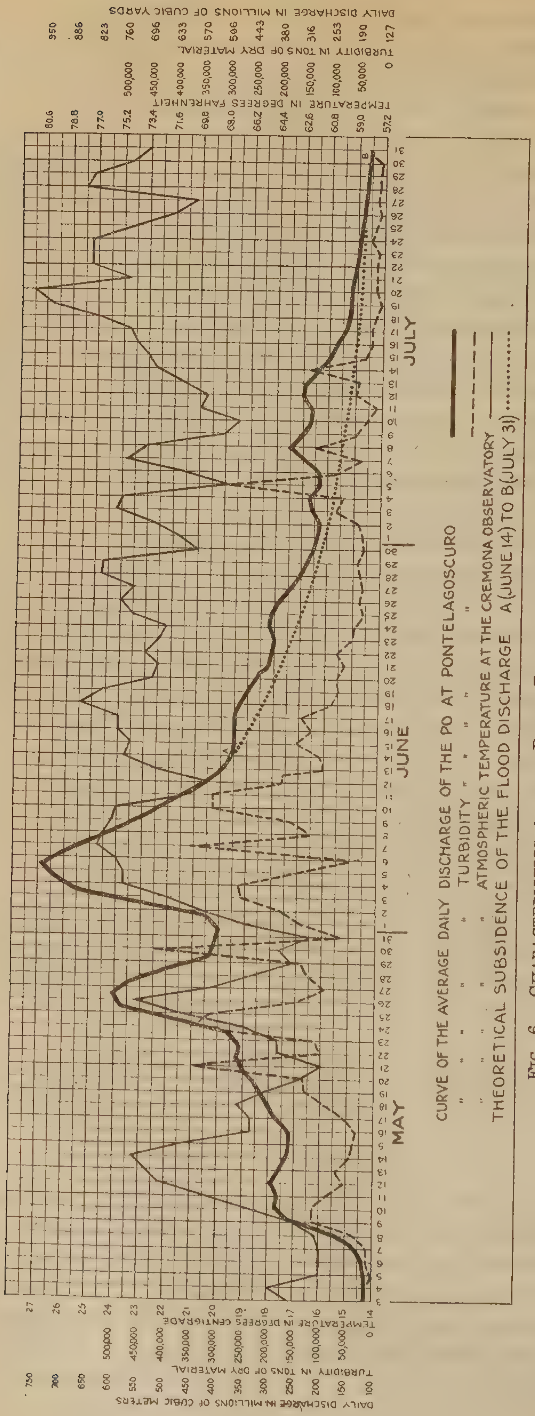

FIG. 6.—CHARACTERISTICS OF THE RIVER PO DURING FLOOD OF MAY-JUNE, 1917.

In Fig. 6 is shown the effect of the rapid succession of three heavy rainfalls which saturated the ground and led to the flood peak of June 1 to 5. It furnishes a measure of the flood in terms of cubic feet per second, while Fig. 7 presents it in terms of elevation of the river surface above the ordinary or low-water height at several important cities. Fig. 3 shows the daily rate of discharge for the hydraulic year, 1923-24, as well as the total number of days in the year upon which the river was above or below a given height or volume.

A glance at Figs. 1 and 2 shows the wonderful profusion of rain-gauges, stations for ground-water observations, river-gauging stations, and the wide variation of precipitation. The yearly range is from more than 120 in. in depth on mountain slopes to less than 20 in. in parts of the broad low valley, a ratio of 6 to 1.

From an inspection of Figs. 1, 2, 3, 6, and 7 it is plain that uncommonly complex problems of hydrology are presented on the River Po. The reports mentioned previously contain many other diagrams of scientific and practical interest. A map showing in detail the successive positions of this river within the historic period of 2 000 years would be of much interest, notwithstanding that for a century or more this stream has been held to the location in which it is now found. The writer knows of no such map, but the story of the control of this river will not be complete without one.

THE DIKE SYSTEM

A different arrangement of the dikes is presented along the lower river, below the ancient City of Ferrara, from that along the Upper Po. Along the lower river reliance appeared to be placed wholly on one main dike on each side of the river, whereas along the upper river there was a network of dikes. Along the Adige River near Rovigo there was a single line of dikes on each side, which appeared to be about 25 ft. high. The proximity of many ancient appearing buildings to these massive dikes indicated they had long held this river to its prescribed course.

At nearly all points seen by the writer, the main dikes of the Po had a top width of at least 16 ft., in other places of 23 ft., surmounted by a well graveled highway, excellently maintained. This appears to be better practice than that which prevails along many of the Mississippi levees, where the roads are kept off the top and placed along the inner terrace or berm, for the reason that water collecting in wheel ruts weakens the levee. Plainly, the remedy is to maintain a good hard-surfaced road width without ruts and with a high crown. Travel along the top of a dike scares away muskrats from burrowing into it.

The slopes on the river side were 1 vertical on 1½ horizontal. At the concave bends this river slope was roughly paved above low water, and was revetted below low water with a loose pile of stone rip-rap a few feet thick; so that in case of undercutting, stones could slide down and protect the base. On the land side, the slopes were 1 vertical on 2 horizontal, with one or more berms 5 to 10 ft. wide, at intervals of about 15 ft. in elevation. The height of the Po dikes above the fields was apparently about 25 ft., at a distance of 125 miles

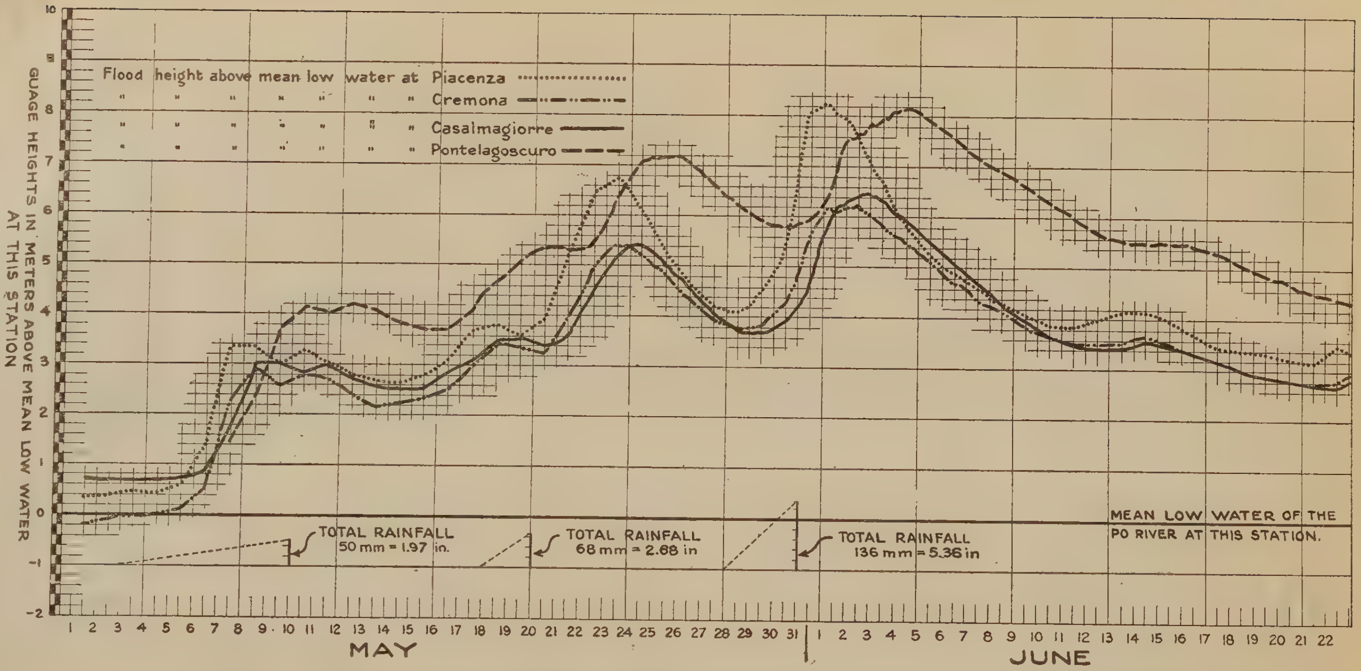

FIG. 7.—FLOOD HEIGHTS AT PIACENZA-CREMONA-CASALMAGGIORE-PONTELAGOSCURO DURING THE PO RIVER FLOOD OF MAY-JUNE, 1917.

from the sea. At some places the height is said to be greater and dikes 45 ft. high are said to exist on some of the tributaries.

Dike at Concave Bends.—Fig. 8 shows a standard cross-section of a sturdy structure, 23 ft. wide on top, well adapted for a highway, and considering the quality of fine sand of which it is built, sufficient in thickness to escape saturation and sliding during the short-lived floods of the Po. The sand of which these dikes are built appears in general to be composed of grains $\frac{1}{50}$ to $\frac{1}{100}$ in. in diameter, through which saturation would proceed slowly; but that they are sometimes rendered dangerous by saturation was indicated by photographs taken during the great flood of 1926 of inshore banks completely covered with a layer of sand bags to stabilize them. This condition, however, was the result of three successive floods within 28 days, illustrated on Figs. 6 and 7.

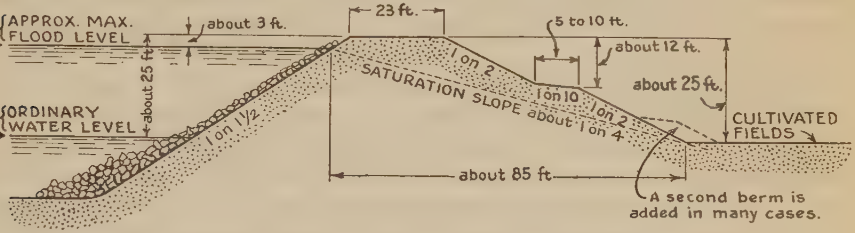

FIG. 8.—TYPICAL DIKE AT CONCAVE BENDS ALONG THE PO RIVER.

The material for dike building so far as noted, was extremely fine sand like that in the river bed and seemed to present no large proportion of slippery colloidal material. The conditions for dike building, as to material and substrata, appeared more favorable than along the Mississippi, with fewer soft swamps or old bayous to be crossed by the dike. Whatever of these soft spots there may have been in the days of earliest dike building, had been obliterated by centuries of intensive farming. Also, the shorter duration of the floods on the Po than on the Mississippi presents smaller chance for percolating water to penetrate through the dike and thereby softening and weakening it.

The only place where sub-surface conditions could be observed was at excavations for a new navigation lock on the river bank at Ferrara, for navigation from the main River Po southward through a canal to the Po di Primaro and the River Reno. Here, the ground for a depth of about 10 ft. below the natural surface was of clean, fine sand that apparently would pass a sieve of 50 meshes to the inch and was tolerably free from organic matter. Beneath this, the excavation showed a darker colored stratum of rather stiff, clay-like material of a consistency so weak that the steel moulds for concrete sheet-piling, apparently 30 or 40 ft. in length, sank rapidly beneath the dead weight of a heavy pile-driver ram, without a blow, to a depth of 20 ft. and were sent to full depth with relatively few 5-ft. drops of the ram.

From all that could be learned in this brief visit the dikes along the Po and Adige Rivers were in far less danger from undercutting and sliding inward during a flood than those along the Mississippi. This might be expected, because of their smaller depth which, in greatest flood, presents a bank-face on the concave shore of only one-half or one-third the height found in many places along the Mississippi. Moreover, the writer judged that along the Po there is an absence of submerged snags or water-logged trees from caving banks to give the current a vicious twist against the bank. Local conditions apparently do not call for a broad protecting mattress on the river bed at the foot of the bank slope as on the Mississippi. Safe resistance to the underscour of the flood currents of the Po, even at the sharpest concave bank, apparently can be secured by a relatively thin breastwork of fascines, pinned down and stiffened by a few rather slender piles, and weighted with stone rip-rap.

A completed training wall of this kind near Casalmaggiore, which successfully withstood the great flood of 1926, was inspected. Another training wall, under construction, about 10 miles up stream from Parma, also was inspected. A cross-section of the dike combined with a training wall under construction about 10 miles northwesterly from Parma is shown in (*E*), Fig. 9.

The writer was told by Senator Luiggi that for a part of the length of the Po, where floods are highest and danger is greatest, there are on each side of the river two main dikes in parallel, at a considerable distance apart, which thus provide a first and second line of defense, such that, if the first line of dike is ruptured, the second will protect the back country. Moreover, the area between the two dikes presents a noteworthy addition to the reservoir capacity of the river. The writer did not happen to see any clear cut examples of this double dike with reservoir space between, but found in the region between Cremona and Casalmaggiore an equivalent protection afforded in the multiplicity of secondary dikes beneath the highways and cross-roads, some of which on curved lines parallel to the river, apparently marked the shore line of centuries ago. (See Fig. 4.) In many places a third lesser dike has been built close to the water's edge by the farmer to permit the utilization of an additional strip of ground, with the expectation that this will be drowned out once every 5 or 10 years. There is, moreover, a system of major and minor dikes along much of the river, as illustrated by Fig. 10.

Everywhere, at short intervals along all the dikes, were broad ramps carrying the road down to the level of the cultivated fields. Some of these main outer dikes present a slightly zig-zag alignment, for which there was said to be some military reason, there being a main highway along the top.

This network of main roads and cross-roads, each on top of an embankment about as high as the main dikes, which were found all along the river from Cremona to Parma, presented a great safeguard against widespread inundation, which was mostly absent down stream from Ferrara to Rovigo, or within the lower delta. Districts along the down-stream portion of the river obviously need less substantial safeguards than midway of the stream, because of their multiplicity of old channels by which the water can escape quickly to the sea.

THE RIVER CHANNEL

The average impression left in the writer's memory, after inspecting the River Po at many points from Piacenza to the sea, at a stage about 3 ft. above

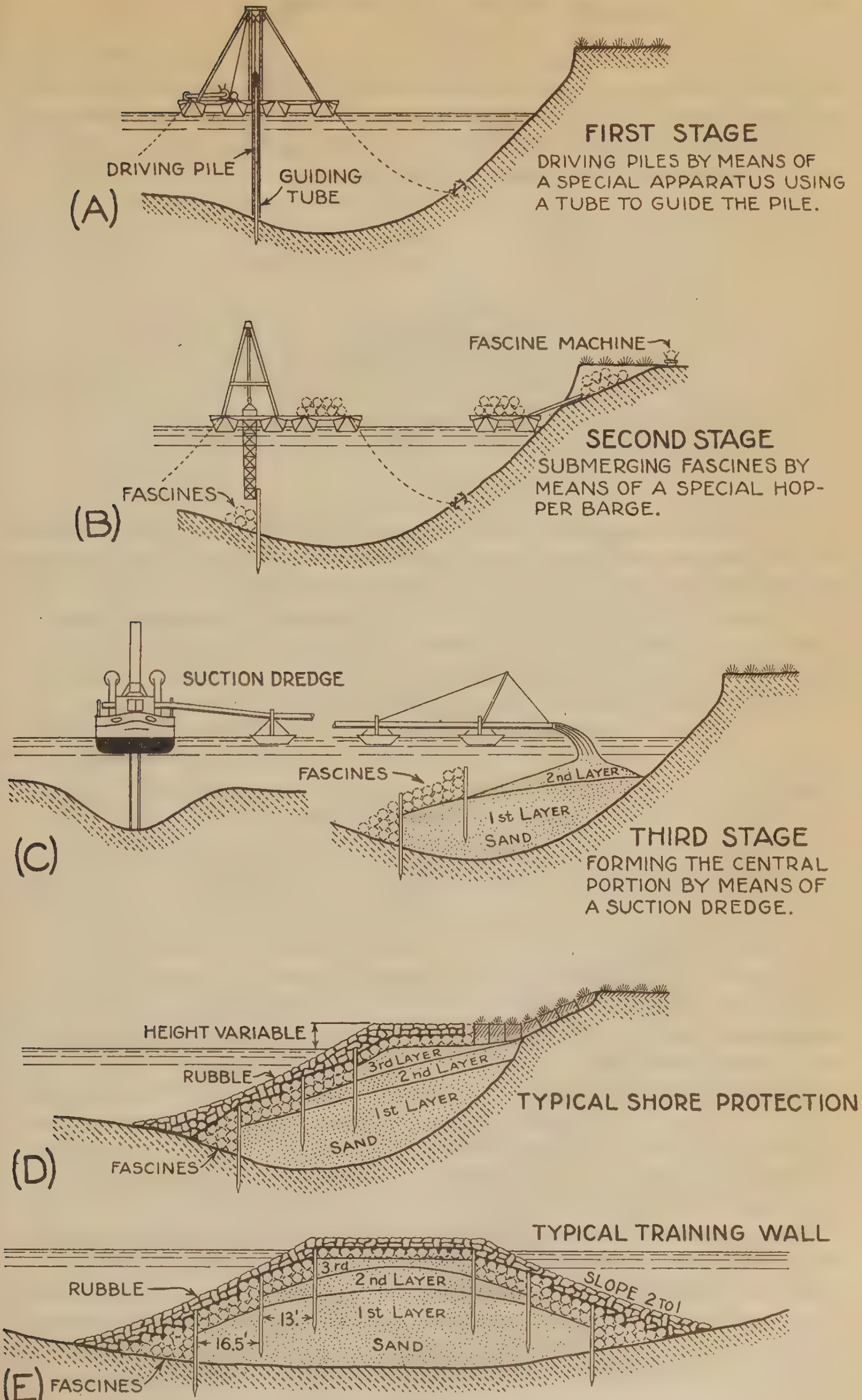
DRIVING PILE
GUIDING TUBE
(A)
FIRST STAGE
DRIVING PILES BY MEANS OF A SPECIAL APPARATUS USING A TUBE TO GUIDE THE PILE.
FASCINE MACHINE
FASCINES
(B)
SECOND STAGE
SUBMERGING FASCINES BY MEANS OF A SPECIAL HOPPER BARGE.
SUCTION DREDGE
FASCINES
2nd LAYER
1st LAYER
SAND
(C)
THIRD STAGE
FORMING THE CENTRAL PORTION BY MEANS OF A SUCTION DREDGE.
HEIGHT VARIABLE
RUBBLE
3rd LAYER
2nd LAYER
1st LAYER
SAND
FASCINES
(D)
TYPICAL SHORE PROTECTION
TYPICAL TRAINING WALL
RUBBLE
3rd
2nd LAYER
1st LAYER
13'
16.5'
SAND
SLOPE 2 TO 1
(E)
FASCINES

Fig. 9.—Bank Protection and Training Wall for River Po.

low water, is similar to that of the Missouri River between Kansas City, Mo., and Omaha, Nebr.; or of a river about 700 to 1 000 ft. wide, with many sandbars and wooded islands, and an almost continuous succession of loops and bends; and with main dikes in some places only ¼ mile apart across the river and 1 mile or more apart in other places.

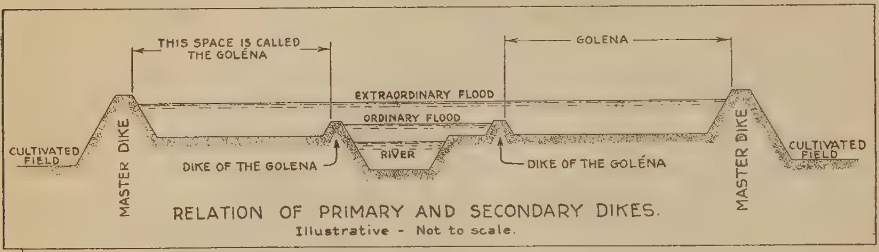

FIG. 10.

The depth at low water is said to average from 10 to 20 ft., with depths of 30 ft. in the pools near concave shores or sharp bends. Below Cremona, the bed shows no gravel and no sand coarser than would go through a sieve of 50 meshes to the inch, except at a few spots, as, for example, at Cremona, where, as at Kansas City, boats reclaim the medium coarse sand from the river bed for cement mortar. The river appeared less turbid than the Missouri, and at low stages had less soft silt at the water's edge.

Frisi's report of 1762 discusses at length the accumulation of gravel on the beds of the tributaries just down stream from where the slope changes from the hill to the valley floor, but states that nothing coarser than sand reaches the main river. It has been reported* that:

> "Between Pavia and Piacenza the nature of the bottom is for the most part made up of gravel or large grained sand. The materials of the river bed are relatively immovable. * * * Excavations made by dredges have lasted in general for many months and in some cases even for years. * * * From Piacenza to Cremona are found the last gravel and the first sands."

So far as could be learned, "cross-over bars" of gravel or coarse sand, are much less pronounced on the Po within its course of small declivity below Cremona, than on the Mississippi River, and its tributaries; but from certain statements in printed reports the writer infers they are not entirely absent. He did not have time to inspect the river and its banks and dikes from the highway bridge above Cremona, but had a brief view of the banks and uncovered parts of the bed at the railroad crossing at Piacenza. A tour along the dike all the way to Turin would doubtless have been instructive, but the writer was interested chiefly in the lower river.

The absence of gravel in the main river below Cremona seems due to the width of the plain back to the base of the steep hills. The chief northern tributaries—Ticino, Adda, and Mincio—flow through lakes, as Maggiore, Como, and Garda, where all such coarser material is deposited. The Apennine tributaries have no such lakes. Nevertheless, apparently, they fail to deliver gravel across the great width of the valley floor to the main river.

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* The report on studies of the Po to the Ministry of Public Works, under decree of July 15, 1922, pub. in 1924 under the title, "Relazione ed Allegati," p. 48.

The proportion of fine silt and of slippery colloidal material in the main river when only 3 ft. above its lowest stage looked rather small, and perhaps this may simplify the problems of safe dikes with rather steep slopes.

These are the impressions left upon a rapid traveler. To answer all questions about bars, bends, caving banks, and stability of channel decisively would take far more time and facilities than were available.

Regarding Navigation

Increase in navigability is one of the chief purposes of the extensive works for river training, recently begun after several years of study, as previously described. Sand-bars in a few places in the lower river and gravel bars in the river above Cremona have presented obstacles on which considerable work by suction dredges has been performed in the past. It is now hoped that, by training the river into a single relatively narrow channel of nearly uniform width, the requirement for dredging will be largely decreased. The report to the Department of Public Works—previously quoted—expresses the hope (pages 48 to 50) that the delivery of sedimentary material into the river will at some future time be lessened

> "* * * by regulation works on the mountain sides, which, while adding to the safety of the habitations, and that of the roads and other public works, will also greatly help in the better maintenance of the depth for navigation by rendering the water less turbid."

Also, this report states:

> "The dredging work may in time be notably reduced where, with the appropriate work of retaining, reforestation, and draining, provision is taken to regulate the mountain basins of the Apennine torrents, works which will undoubtedly result also in the efficient consolidation of the landslides which, especially in the Emilian Apennines, continually menace the roads and habitations."

Fifteen miles up stream from Casalmaggiore, the writer saw 600-ton barges laden with rip-rap from the hill quarries north of Padua, which he was told had navigated the river up stream from the canal lock about seven miles easterly from Adria without difficulty. No other boats of importance were noted except some fishermen's boats near the mouth. Evidently, navigation of the Po has not yet become active. Perhaps it awaits improvement at the lower mouth.

At the delta mouth there is a sudden shoaling from the ordinary depth of 10 to 20 ft. to from 2 to 5 ft. at low water for about ½ mile in length of course. From this bar, under present conditions, navigation for barges with drafts of 7 or 8 ft. may now be readily accomplished.

In the writer's opinion this shoaling at the exit doubtless is caused by the precipitant action of the sea water on the fine sediments which are carried along readily in fresh water by moderate currents.

Frisi and other writers, of from 100 to 200 years ago, mentioned shoaling at the mouths of other chief Italian rivers, and presented various theories as to its cause. The favorite explanation had to do with drift, under the action of littoral sea currents, of sediment brought down by the river.

River Training in Progress on the Po

As a part of the recent widespread industrial activity in Northern Italy, river training has been begun for developing navigation on the Po and its tributaries. Connection by locks is to be made with the canals connecting with the Adige and the Reno to ports on the Gulf of Venice and on the Adriatic Sea, thereby serving the river cities all the way up to the great industrial districts of Milan. These works are designed to provide for steel barges of 600 to 800 tons capacity and of 8 ft. draft. The plans comprise, chiefly, training of the river into a curving channel of nearly uniform width of about 1 000 ft., with a narrowing at points of contraflexure instead of an expansion, such as is found in many places along the Mississippi, and which extra width, as explained by the late James B. Eads, M. Am. Soc. C. E., may be largely responsible for its bad cross-over bars.

The regulation of the Po is designed to give ultimately a width of 300 m. for the bends and only 250 m. for the transition sections between the bends.

This great public work was actively begun about 1926 after years of patient study and planning under Professor Giandotti and others. It is understood that the present plans are tentative and subject to modification as the work proceeds. In brief, the system is designed to make a limited amount of money go as far as possible with safety. It is proposed to protect and hold most of the concave shores in their present position by a revetment composed of fascines and rip-rap as outlined in (*D*), Fig. 9, without any mattress work. Where the river has to be narrowed at a wooded island or a "middle ground", training walls will be built of a type similar to that shown in (*E*), Fig. 9. The lesser depth permits these structures to be of much simpler character than has been found to be permanently reliable on the Mississippi.

It is hoped by those in charge that the total length of protection, by means of fascine and rip-rap like (*D*), Fig. 9, which will ultimately be required, will probably not exceed one-fourth the entire length of the river bank. This limited application of bank protection is obviously a matter of great economy, and it is expected that by holding the river thus narrowed to a uniform width, somewhat rigidly to its present curving course, in which it has long seemed content, and retaining its present length, elevation, and slope, that the Po will be satisfied to remain within the path now laid out for it, with protection only along the concave shore. The writer understands that the uniformity of width, with shore lines at the bends curved somewhat precisely to prescribed radii, all as shown on certain of the plans portions of which are copied on Fig. 5, is not expected to be produced forthwith, but is the ideal shore line toward which all the revetment and training walls will tend.

This retention of the present total curvature leaves the river 50% longer from the Adda confluence to the confluence with the Secchio, 82 miles down stream, than if the bends were cut out; in which case doubtless both banks would have to be reveted most of the way. Here and there, where the river shows a tendency to dig too deeply into a concave bank and thus threaten the adjacent dike, one or more spurs of fascines and rip-rap have been built out at a slight angle to the current, to turn it back toward mid-stream. The success with which the dikes, remarkably close to the edge of the low water, are maintained by light rip-rap, and by protecting only the concave bank, indicates that the Po is far less restless within its banks than the Mississippi, and that its subsoil presents no such treacherous conditions as those which without warning caused the crevasses at Poydras and Wecama, La., and threats of crevasses at Tunica, Miss., Stanton, Miss., etc., during the Mississippi flood of 1922. There may have been difficulties of which the writer did not learn. The few days spent along this river were far too brief for studying all its possible vagaries.

It was of special interest that engineers of great experience and manifest ability, like Giandotti, Fantoli, Luiggi, Mauro, and others of eminence, had evidently concluded that they could safely devote the limited funds available for improvements for navigation and flood control on the 82 miles of the River Po between the Adda and the Secchia, to confining it to a uniform width of about 825 to 1 000 ft., following nearly its present channel, without important change of total curvature, without making cut-offs across bends for straightening, and with a total length of special bank protection and revetment amounting to only about 25% of the bank. The projected channel section is shown in Fig. 11.

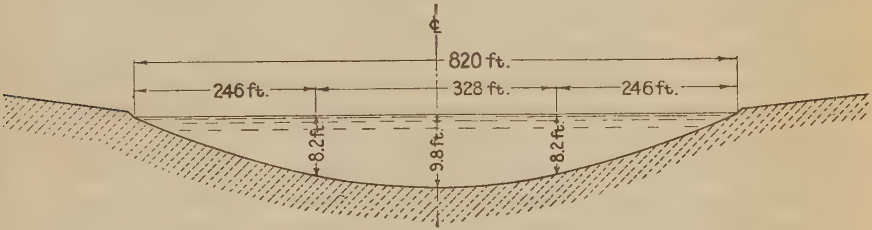


FIG. 11.—PROPOSED NORMAL SECTION OF REGULATED CHANNEL FOR THE PO.

The writer would have favored a more radical straightening and cutting out of bends, beginning near the mouth, thereby lowering the flood heights on the upper river and shortening the path for navigation, all, of course, subject to study and experiment in the field and in a hydraulic laboratory. Such a radical innovation would have cost more for the early stages and would have delayed the opening of systematic barge navigation. The method adopted is in the nature of "playing safe" and minimizing the immediate outlay.

In some reaches within this distance the space between the main dikes is now about a mile in width. Within this space the river has been free to meander, and alongside its main channel there have been deposited banks of sediment to such a height that the shallow depth over them in a great flood adds relatively little to the discharge capacity. The controlled narrowing to a uniform width of 825 to 1 000 ft. will add largely to land available either for cultivation of narrow forests of quick growing pulp wood, or for field crops enclosed by cheap low dikes that may be flooded once in 10 years. The present main dikes, farther back, will remain as a second line of defense.

In the early stages of improving conditions for navigation, much dredging has been done at shoal places; but the writer was given to understand that, in the future, it is hoped to avoid any large amount of dredging, except at the outlet bar where fresh water and salt water meet, the current, as guided by the new training walls, being expected to shape the future channel.

Professor Giandotti, and others, stated that they were basing the designs on intensive studies of the river itself, its actual depths, and its tendencies at various localities, rather than on experimenting toward any radical changes in curvature or in design of groynes or revetment forms by means of laboratory models. This is in marked contrast to the present practice of river engineers in Germany. This attempt to confine a great meandering river to a constant width with such a minimum of shore revetment at a minimum of cost is heroic and so far as the writer knows is unprecedented.

Notwithstanding they have been guided by centuries of observation on the behavior of this river and its neighbors, in developing their designs, the writer is confident that an adequate hydraulic laboratory could be of great aid and lead to great economies. They have a long way to go, and the proposed laboratory at Stra may yet be of service in their work.

In Figs. 4 and 5 are shown plans of short typical reaches, copied from maps on larger scale covering about 100 miles of the river's course that recently has been studied in much detail. The relative positions of the present main channel, the islands, sand-bars, and side channels are typical of all sediment-carrying rivers through a nearly level country.

The proposed regulated channel is indicated by dotted lines, which show a narrowing at reversals of curvature between the bends. All the water is to be concentrated in one channel, by stopping off the lesser branches of the "braided stream" at their points of divergence.

Bank revetment is indicated where the channel is close to the main dike and at all concave banks of bends. In many places the location of the main dike bears little relation to the present main channel. The width between them varies (Fig. 5) from hardly more than ¼ mile near the confluence of the Mincio to 2½ miles at a location 6 miles up stream. Since the ground for miles around is an alluvial deposit of substantially the same texture, these relative locations of channel and dike are accidental and matters of ancient history.

Similarly, Fig. 4 shows details of the position of spur-dikes for confining the water to a single channel in a typical section. The current "retards" are simple lines of piling, against which floating rubbish lodges in time of flood, thus retarding the current and inducing deposits of sediment. As stated elsewhere, it is found necessary to strengthen the up-stream retards by cross-bracing, and, in some places, by spur-dikes armored with stone rip-rap. These pile retards are in purpose very much like the permeable dikes tried unsuccessfully on the Mississippi River many years ago.

Brief Extracts from Report of March, 1924

In the report dated March 17, 1924, to the Minister of Public Works, Gabriello Carnazza, Deputato al Parlamento, by Engineer Leone Roman-

Jacur, Senatore del Regno, on the improvement of water communication from Milan to Venice, which makes, with its appendices, a volume of 236 pages, there is much of interest to engineers. Space permits only very brief extracts. On pages 45 to 51 are a series of questions and answers from which the following positive statements are condensed:

(1) The pile retards exposed to strong currents have suffered injury, and for permanence require the protection of rip-rap. Those not exposed to strong currents have endured.
(2) The effect in increasing depth of channel up to that time, when only a small part of that proposed had been installed, had not been specially noteworthy.
(3) The safety of the main dikes had not been impaired by the obstruction to river flow caused by the pile retards.
(4) From this experience it was deemed advisable to extend the works, but to limit the simpler form of pile retards to places not exposed to strong currents. Experience on the Tiber, Reno, Elba, and Vistula Rivers gave hope of success on the Po.
(5) The recent studies showed an average discharge of sediment carried by the Po, of about 10 000 000 cu. m. per year. (Equivalent to about 8 300 acre-ft., or 13 ft. in depth on 1 sq. mile.)
(6) This amount of material on its way to the sea impedes navigation by forming sand-bars where the river is broad at time of low water; and there are excellent reasons for expecting improvement to follow the proposed narrowing and the consequent increase of velocity, at low water.
(7) Until means were lacking for confining the river, dredging was necessary. This was begun in 1910.
(8) The permanence of cuts made by dredging is dependent on the material forming the river bed. There is a transition zone between Piacenza and Pavia, from gravel to fine sand, where the surface of the river bed changes.
(9) Dredging, in the course of time, may be greatly reduced, particularly after reforesting the basins of the Apennine torrents. Works for regulating these torrents and preventing erosion along their course will greatly aid the works of regulation along the River Po.
(10) From the studies of more than a century, the bed of the River Po presents no tendency to progressive rising. The sediments brought in by the torrents merely form temporary obstacles to navigation. Dredging will rapidly remove these obstacles, and, in course of time, after regulation and narrowing, the River Po may be expected to clear itself.

On pages 60 to 62, the conclusions of the Commission for various reaches of the river, are summarized as follows:

(1) From Muncio to Carnavalla-Po, a small amount of dredging suffices to maintain channel depths for 600-ton barges save at a few places presenting exceptional conditions; here regulation works are needed. For the Adda-Muncio Tract, regulation works are needed to supplement dredging.
(2) Trials on the most difficult section of the River Po, from Taro to Costello, instituted first of all from theoretical considerations, and without previous experimental data, seem to have been not altogether successful, but to have pointed the way toward improved structures promising success.

(3) The conclusion seems to have been reached by the Commission, that navigation interests could be better served by regulating the Po for barges of 600 tons capacity, than was possible by means of a parallel system of canals, which would accommodate barges of only 250 tons, and that the river can be made a fine waterway from Venice to Milan similar to the principal rivers of France and Germany.

In pages 81 to 85 there is presented a summary of the study of the regulation of the Po at low water, by the Hydrographic Office of the Po. Many of the facts therein stated have been presented in the preceding pages. The following, however, seems worthy of special attention at the risk of repetition.

The tributaries of the Po comprise three groups of very different characteristics:

(1) Torrential rivers from the Apennines, coming from the south in rapid impetuous floods, carrying much sediment.
(2) Torrential rivers from the Western Alps, similar to Group (1), but with smaller precipitation and, therefore, bringing less sediment.
(3) Those from the Alps, coming from the north, which flow through lakes and, with the exception of some small tributaries, deliver water free of sediment.

The floods in these groups come at different times, those from the Apennines first. The River Po commonly presents two low-water stages. The earlier Apennine floods leave obstructing sediments to be scoured out by the later floods of less turbid water from the north.

Cross-sections measured 10 years ago show no noteworthy change in area for low water, from those measured 50 years ago.

Measurements of average low-water elevations carried on throughout a century at Pontelagoscuro show undulations, but neither a progressive rising or lowering, thus proving no important change in elevation of river bed due to sediments unless it be that the climate and run-off have changed, which records disprove.

The turbidity measurements in the 7 years, 1914 to 1922, indicate an average annual transportation of 23 380 tons, or 17 450 cu. m. of silt, equivalent to 14 400 acre-ft. per year. (Note the estimate on page 187 of 8 300 acre-ft.)

Lombardini, more than a century ago, from the advance of the delta and the depth of deposit, estimated 27 000 000 cu. m. per year, of sedimentary material brought down by the Po, equivalent to 22 000 acre-ft. per year.

Marinelli from delta extension, from 1823 to 1893, estimated 29 000 000 cu. m., equivalent to 24 000 acre-ft. per year, as an average for the previous 70 years.

Also, some interesting details of the progress of the regulating works are given (pages 205 to 217 of the report just quoted) which are abstracted as follows:

The Minister of Public Works (Director General of Hydraulic Works) ordered the superior officers of the Civil Engineering Corps at Parma to present projects for increasing the navigation of the Po so as to permit the passage of barges of 600 tons at low water from the confluence of the River Adda to that of the Carnavalla-Po. The general plan is to unite all the low-water flow in a single channel with an area of cross-section sufficient to avoid too high a velocity of current for convenient navigation, while high enough to prevent obstruction by deposits of sediment.

The low-water discharge for planning the works was assumed to be 400 cu. m. (nearly 1 500 cu. ft.) per sec., and the slope, 0.00018, or 0.93 ft. per mile in the 26 km. (16 miles) from the confluence of the Taro to that of the Erza, and 14 km. (9 miles) farther to that of the Crostolo.

One basic criterion of design was that the equilibrium of the river must not be disturbed at either high or low water.

In order that discharge at high water may not be hindered so as to raise the flood level, the new works were all to be designed with their tops not far above the low-water elevation.

In order to avoid changing the low-water regimen of the river, the regulated channel has been designed to have an alignment near that of equilibrium, as shown by the course of the low-water channel in recent years.

In prescribing the shape of the succession of curves and the straight reaches between them, attention was given to the rule deduced by French engineers, that the greater the regularity and continuity in varying the curvature of the concave bank, the greater the regularity of variation in depth. A parabolic curve was adopted as offering, at point of tangency, less discontinuity than a circular path. A surface width of 250 m. was adopted for the straight courses and increased to 300 m. at the apex of curves to provide for deposits of silt at the convex shore.

Two types of works were adopted: (1) At concave shores, protection works and revetment; and (2) for contracting and concentrating the low-water current, systems of piling retards and groynes and rip-rap are to be constructed in the convex and straight portions. These are designed to increase the deposits of sediment and ultimately to close all minor channels.

It was recognized that works directly opposed to strong currents, would have to be made of greater strength than works in more quiet waters, the piling at such places being strengthened by ties between them.

Construction was begun on the current retards during periods of low water. These consist of piles about 8 in. in diameter, 18 ft. long, sunk about 12 ft., with their tops slightly above mean low water.

Lattice work was constructed between the piles at first, but its use was abandoned later, because of the belief that at time of flood, the drift of weeds, etc., would lodge on the piles and serve the same purpose. Later, however, in order to hasten the deposit of silt, the use of lattice work was resumed.

Various directions of the lines of piling in relation to direction of current were tried. Trials showed that the piling was not injured where currents were sluggish, but that strong support for it was needed against swift currents. In general, methods and materials previously used in these localities were adopted as far as practicable.

Costly excavation by mechanical means was avoided, by inducing the current to do the work, until the desired position for the concave bank was attained.

The works completed in 1922 and 1923 and up to the date of this report (1928), comprise but a small portion of all that is contemplated. Progress has been delayed by lack of funds, but the observation of the behavior under stress of the great flood of 1926, of the works already constructed, gives valuable data for future work and so far as the writer could learn this experience is favorable to the methods originally proposed. The future progress of this work and its success are matters of great engineering interest.

Navigation Locks

A very interesting two-part navigation lock of recent construction, about 450 ft. long and 33 ft. wide, with a minimum depth of 12 ft. on the sill, was inspected on the north shore of the River Po about 8 miles easterly from the ancient City of Adria (which gave its name to the Adriatic), and 1 mile westerly from the Etruscan coast line of about 3 000 years ago that is marked by sand dunes. This lock permits ready passage of large barges from the Po, whatever its stage of flood or drought, to the great canal system leading westerly to Adria, northward to Padua and Venice, and eastward toward the sea by the abandoned Po channel of 350 years ago.

The dikes here are 23 ft. high, and this lock provides for 20-ft. lift into the Po at times of great floods. A lift of only about 5 ft. was in evidence at the time of the writer's inspection, with the Po 2.5 ft. above its lowest stage. This lift practically measures the fall in river surface of the Po in the 23 miles from this point to the Adriatic, and indicates an extreme flood slope within the Po, from this lock to the sea, of about 1 ft. per mile, and a low-water slope of roughly 0.10 ft. per mile through the lower delta.

Another lock now being built near Ferrara on the south shore, already mentioned, will give navigable connection to the River Reno.

There is a wonderful development of irrigation canals many hundreds of years old within the northern half of the valley floor. Many of these are fed from the Adda River, and interlock with navigation canals accommodating small craft, and which, in turn, interlock with main drainage canals, all in a way which the writer made no attempt to trace. Plainly, this valley was the ancient homeland of hydraulic engineering.

River-Structure Laboratories

Inquiry and brief inspection of the technical schools at Milan and Padua indicate that the hydraulic laboratory idea has not yet taken an important part in Italy in the problems of river training. This, in the homeland of the "old masters" of hydraulic science, and coming immediately after a visit to ten of the leading hydraulic laboratories, and conferences with a dozen or more of the leading hydraulic engineers, of Germany and Switzerland, seemed unfortunate, in view of the great activity in development of water power and in the reclamation of salt marshes that is now (1928) going on in Italy, and the evident present deep appreciation of scientific education as a foundation for industrial development.

The writer inspected the magnificent new buildings constructed and equipped mainly from funds contributed by the industries and financial institutions of the Milan District for the Engineering College at Milan designed to accommodate 2 500 students. He also inspected the Engineering College at Padua which has accommodations for 1 000 students. At both institutions the hydraulic laboratories have small floor area and evidently are designed chiefly for use in undergraduate instruction. Those at Milan and Padua are excellent for that purpose, but they are not adapted for wide application of the doctrines of dimensional analysis, by means of small-scale models, to the great special problems of water power development, river training, and harbor design, like several of the hydraulic laboratories in Sweden, Germany, Austria, Switzerland, etc., where the designers of high dams, sluice-ways, and harbor improvements are bringing their practical problems to the laboratory in increasing numbers year by year. That the hydraulic laboratory idea is now developing in Italy is proved by the elaborate report on hydraulic laboratories in other countries published about 1925 by Professor Ettore Scimini, of Padua. The writer was told that large laboratories had been delayed by lack of funds.

It is planned to build an excellent hydraulic laboratory at Stra, not far from Padua. Doubtless this laboratory will be at work helping to solve the problems of the Po long before the present extensive program is completed.

Improvements at the River Mouth

Some particularly interesting jetty work, recalling the Eads jetties at the mouth of the Mississippi, has just been begun for creating a navigation channel 12 ft. deep by 1 000 ft. wide, at the northern, principal entrance of the Po to the Adriatic. The depth and position of the mouths of the Po have changed greatly during the historic period (Fig. 12). At some early, perhaps prehistoric, time the main river doubtless discharged to the northeast toward Venice. About 2 000 years ago, the river easterly from Ferrara turned southward through a channel the remains of which is called the Po-Morto, or "Dead Po", and for many years was depositing its sediments in the lagoons at the northern end of the Gulf of Comacchio where maps show many remains of ancient channels diverging from the present river not far from Ferrara. Later, the river turned northward. About 300 years ago, the commercial interests of Venice feared that the mud brought down by the Po and being deposited in the Adriatic by this northern exit, would threaten the depth of navigation available to the ships of Venice. Therefore, they closed this north-flowing principal mouth. This closure turned the chief discharge of water and sediment to the east, not to the extreme south as of old, which perhaps they had expected. The channel improvement by jetty and dredging is at this present principal mouth. This is said to carry 40% of the entire discharge, the remainder being divided through several smaller mouths farther south (Fig. 12), that appear to show no tendency to enlarge.

Although the average depth of the Po for many miles up stream from the bar is said to be from 15 to 20 ft. at low water, for a considerable width of the channel, with greater depth in pools at concave bends, the channels at the junction of turbid fresh water with salt water suddenly shoal to a depth of only 2 to 4 ft. They also spread out over the bar to a much greater total width than up stream. This shoaling extends perhaps half a mile or a mile along the river's course, with very gentle slopes in both directions to and from the crest of the bar.

For defining the new channel across the bar, one long, straight training wall somewhat similar to that shown at (*E*), Fig. 9, but with a larger proportion of rubble stone, is being built at a slight angle to the approaching current, the impact from which should induce a deep channel near the wall. Several long, narrow cuts have been dredged across the bar within the future channel in order to determine its composition and hardness, and perhaps to invite erosion.

There is a strong hope that the construction of a second parallel training wall can be deferred, although the plans provide for it. Later, it may be found expedient to build obstructions across some of the other mouths of the Po so as to force a greater current through this principal mouth, but this is now believed improbable, because for some years past the present chief outlet has shown a tendency to increase in size.

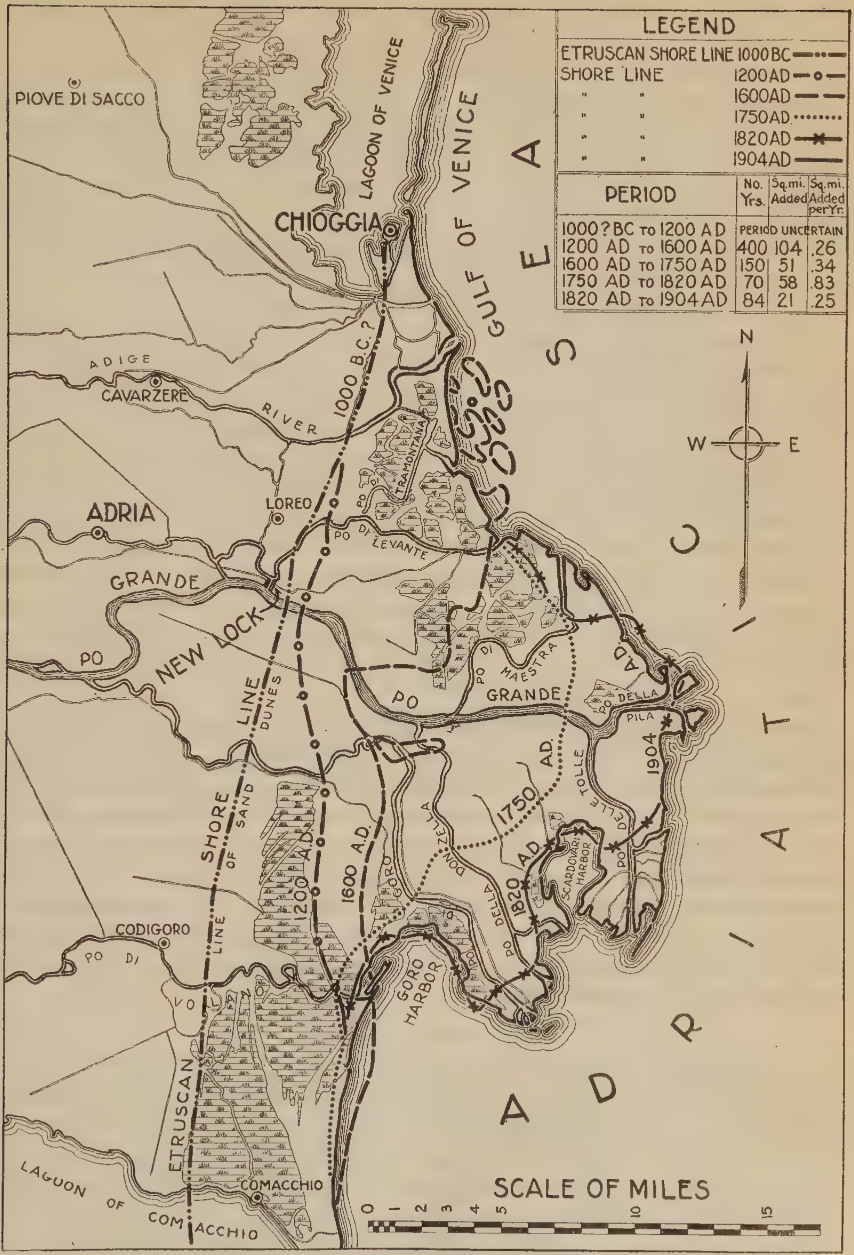
Advance of the Po Delta

The steady advance seaward of the shore line of the Po Delta into the Adriatic during the past 2 000 years to the extent of more than 15 miles at the outward corner of its fan-shaped delta, which is about 50 miles broad from north to south, is both of historic and practical interest (see Fig. 12). To the writer this increased length of river appears to be an important factor in raising the bed and the low-water surface of the Po in its winding pathway through the lower valley, probably more important than any deposit of sediment caused by confining the floods between dikes. At a point which 1 000 years ago marked the river mouth at sea level, the river surface at times of high flood now stands 10 to 15 ft. higher than before and this must affect the height of the river bed and the height of the water surface far up stream.

A rough measurement of the effect of this pushing out of the delta in raising the elevation of the water surface, and perhaps, also, that of the river bed, was found in the reading of the Po River gauge at the lock near Donada, described previously. This stood at the equivalent of 4.3 ft. on a gauge said to have its zero at sea level which, neglecting tides, indicated a fall or slope of 4.3 ft. in about 20 miles of river seaward, while the discharge was such as to raise the river here about 1.3 ft. This indicates a low-water slope of about 3.0 ft. in 20 miles, a large part of which may have been over the bar at the river's mouth.

The newly made delta surface between the mouths appeared in general to be 1 to 1.5 ft. above sea level, explainable by the deposit being largely made from water spreading out in time of high flood and being caught in the grass and bushes of the marsh. A low reclamation dike is being thrown around the newly formed land. For the past two or three centuries the accretion has averaged about 300 acres per year.

Successive advances of the shore line are shown on the accompanying map, Fig. 12, traced from a recent study given the writer at Venice by Luigi Miliani, Director of the Works for the Lower Po. The shore line of the Adriatic in the days of the Etruscans, about 3 000 years ago, is marked by a long north-south line of low sand dunes rising 10 to 20 ft. above the level of the plain. The writer visited these dunes but found no reason that explained why they marked the shore line of that particular stage of development, and are not found along the later stages of its advance, other than that these sand dunes may have been the product of winds blowing landward across a sea beach which maintained a nearly constant position during many centuries while the river was discharging southward through the Po di Primaro, filling with sediment an ancient indentation of the coast at the peak of the Gulf of Comacchio.


LEGEND
ETRUSCAN SHORE LINE 1000 BC
SHORE LINE 1200 AD
" " 1600 AD
" " 1750 AD
" " 1820 AD
" " 1904 AD
PIOVE DI SACCO
LAGOON OF VENICE
GULF OF VENICE
CHIOGGIA
SEA
ADRIATIC
ADIGE
CAVARZERE
RIVER
1000 B.C.?
TRAMONTANA
PO DI
LOREO
ADRIA
PO DI LEVANTE
GRANDE
NEW LOCK
PO
LINE DUNES
PO DI MAESTRA
PO GRANDE
PO DELLA PILA
A.D.
1904
TOLLE
PO DELLE
SCARDOVARI HARBOR
1750 A.D.
1820 A.D.
PO DELLA DONZELLA
GORO
1600 A.D.
1200 A.D.
SHORE LINE OF SAND
CODIGORO
PO DI VOLANO
GORO HARBOR
ETRUSCAN LINE
COMACCHIO
LAGOON OF COMACCHIO
N
W
E
SCALE OF MILES
0
1
2
3
4
5
10
15

PERIOD	No. Yrs.	Sq.mi. Added	Sq.mi. Added per Yr.
1000?BC to 1200 AD	PERIOD UNCERTAIN		
1200 AD to 1600 AD	400	104	.26
1600 AD to 1750 AD	150	51	.34
1750 AD to 1820 AD	70	58	.83
1820 AD to 1904 AD	84	21	.25

FIG. 12.—GROWTH OF THE PO DELTA.

The tides in the Adriatic, other than wind tides, are only about ½ ft. in range. A wind tide may lift the water as much as 5 ft. The pavement in the Piazza San Marco, in Venice, is said to have been flooded to a depth of a foot after a long-continued, strong, northward-blowing wind.

Extensions of land into the sea at the mouths of other Italian rivers are briefly discussed by the old authors who sought, in littoral currents, explanations for the direction of drift. Frisi found this principally in Tuscany, Romagna, and La Marca. The ancient Port of Pisa is at present 10 miles distant from the sea. The City of Ravenna which, under the Roman Emperors, was celebrated for its fine harbor on the Adriatic, is now 5 miles inland. Some of these ancient authorities concluded that to insure the safety of a sea harbor a sediment-bearing river should be diverted to enter the sea at a point at least 8 miles distant from the harbor entrance.

Vast areas of shallow lagoons and low-lying mud banks in the outer delta have been reclaimed within the past century by dikes and drainage. This reclamation is still being pushed outward. There is nothing now that looks like a marsh, except a narrow fringe near the outer edge.

The treatises of Frisi and others indicate that many swamps and shallow lakes like that now around Mantua, which existed midway of the valley far up stream from Ferrara a few centuries ago, were drained by canals running to points down stream along river or seashore at a lower elevation. The providing of the present intercepting channel for the many north-flowing streams easterly from the Reno, by taking the Reno into the Po di Primaro, much as proposed by Frisi about 1760, appears to have greatly facilitated the reclamation of the vast area of swamps and shallow lagoons, or "valles", southerly and easterly from Ferrara.

Extensive reclamation in the lower part of the Po Valley where the land is now below sea level, is said to have awaited the development of steam pumping machinery. In the upper valley swamps could be drained by gravity. As compared with Holland, Italy appears never to have made large use of windmills for pumping from submerged ground. Large, old, shallow lagoons between Ferrara and the sea have been drained within the past 50 or 75 years, rivaling the Haarlemmer Meer in area and fertility. These reclaimed lands have been so smoothed and covered with verdure that the fact that they were shallow lakes not long ago is not apparent to the casual traveler on the excellent roads which cross them.

LAND BELOW SEA LEVEL

Along the lower reaches of the Po and the Adige Rivers, in the vicinity of the ancient shore line and the lock previously described, west from the ancient sand dunes, are many thousands of acres of intensively cultivated land lying at a level of from 1 to 4 ft. below mean sea level. The writer infers that this low level represents the bed of old lagoons which have been diked and pumped out and thus reclaimed from the sea, like the bed of the Haarlemmer Meer in Holland. The ancient maps, for example that of Manfredi of 200 years ago, show many vast lagoons not noted on modern maps, but there are still many of these lagoons awaiting reclamation around the Gulf of Venice and the Gulf of Comacchio.

Local subsidence of the earth's surface as a cause for depression of these lagoons was suggested by authors of one or two centuries ago. Frisi, in his fifth chapter, discusses this in terms of the rising of the waters rather than of subsidence of the land. He says the continual rising of the waters was not unknown to the learned of the Sixteenth Century. Eustace Manfredi established the fact at Ravenna, a few miles south of the mouth of the Po, by levels on floors of several ancient edifices which he found below the level of the sea. Bernadin Zendrini confirmed this at Venice, where rings formerly used to fasten boats are now below the level of the sea, and the subterranean church of St. Mark is no longer serviceable. Lest this be thought to be due to settlement of the foundations, he cites observations on the opposite side of the Adriatic, where the sea is now higher than the floors of ancient buildings built on solid rock.

Naturally all this was of interest to the writer because of his recent investigation of earth tilt in the Great Lakes region in the United States and his discovery of 25 years ago, from old bench-marks and tide records, that subsidence at Boston, Mass., had been going on at the rate of about 1 ft. per century;[*] also, because of finding in the course of studies for the New York water supply, about 23 years ago, that the tide-gauge records near New York, N. Y., indicate subsidence at the rate of about 0.6 ft. per century.

River Bed Higher than Adjacent Land

At the beginning of this paper the oft-repeated statement that the elevation of the bed of the River Po is higher than the land on either side, was mentioned. It is stated in the Encyclopædia Britannica that:

> "Owing to its confinement between these high banks and to the great amount of sedimentary material which the river brings down with it, its bed has been gradually raised so that in its lower course it is in many places above the level of the surrounding country. A result of confining the stream between its containing banks is the rapid growth of the delta."

The writer was unable to learn of any valid complaint against the dike system of the River Po on this score. The statement appears to be untrue.

The report to the Ministry of Public Works, previously quoted, states most distinctly, on page 50:

> "As seen from the said studies of the Hydrographic Office of the Po and from the hydrometric observations of the Po for a century, it is evident that the bed of the river presents no tendency to a progressive rising."

It is probable that on the tributaries of the Po, at the outer edge of the nearly level valley floor, immediately down stream from the steeper gradient of the river within the hills, there may be important deposits within the dikes which have raised the beds of these tributaries above the level of the land outside the dikes. The writer has seen examples of this on streams in Korea, but in this brief inspection no place could be found where the bed of the Po was at a higher level than the banks outside, nor could it be learned from the engineers of any such place along the main channel of the Po down stream from Cremona. Professor Scimemi, Chief of the Department of Hydraulic Engineering, in the Technical University at Padua, has written that he knows of no such instance from Mincio to the sea.

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[*] See Appendix No. 20, Report by John R. Freeman, Past-President, Am. Soc. C. E., to the Committee on the Charles River Dam, Commonwealth of Massachusetts, 1903.

[†] See p. 653, Report of the Commission on Additional Water Supply for the City of New York, 1904, by William H. Burr, and the late Rudolph Hering, Members, Am. Soc. C. E., and John R. Freeman, Past-President, Am. Soc. C. E.

On the Adige River and other streams immediately north from the Lower Po, there are places where the bed is higher than the adjacent land. The Adige, unlike the Adda, Ticino, and Mincio, has no large lakes in its course. A cross-section, traced for the writer in the engineering office at Rovigo, is presented in Fig. 13. This section is about 22 miles up stream from Rovigo, or about 55 miles up stream from the mouth of the river. It shows the river bed 2.3 ft. higher than the land outside, and the water in the river 22.3 ft. above the land. Presumably, this was at ordinary stage, and flood stage was only 2 or 3 ft. below the top of the dike. In some other rivers, the bed is several meters higher than the land outside.

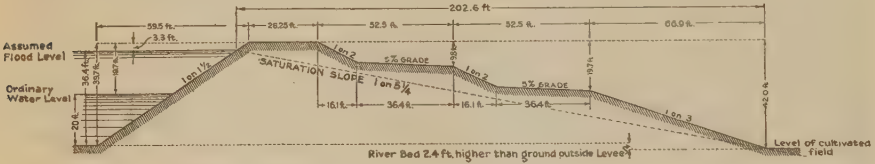

FIG. 13.—TYPICAL SECTION OF LEVEE ON RIGHT SIDE OF THE ADIGE RIVER.

On the Reno River where it leaves the steep slope within the hills southwesterly from Bologna, one would probably find similar examples. Time did not permit a tour along both the Reno and the Adige. The writer is confident that tracing these rivers from the sea, back into the slopes within the foothills, would disclose much of great interest to an engineer.

In closing this narrative it is suggested to the engineer tourist that, in addition to the art treasures of Florence, the marvels of cathedral architecture, or the mysteries of the Etruscan hill-cities, he can find in Italy plenty, both ancient and modern, that is of surpassing interest in the line of his own profession. The books of the "Old Masters of Hydraulics" also are fascinating. One who has scant knowledge of the language can translate much from analogy to the French and Latin of schoolboy days. The writer is trying to enlist the aid of some Italian engineers, including Lorenzo Allievi and Gaudenzio Fantoli, in preparing abstracts of some of these old treatises, which will trace the development of this branch of science and art in its early home, for the benefit of English-speaking engineers. The Italian backgrounds of hydraulic engineering present some very interesting scenery.

All that was seen and heard, of these works of river improvement, reclamation of marsh lands along the Po and farther south, and of the great activity in hydro-electric development of the highest order of excellence on Alpine streams, as well as the enterprise in technical education, the encouragement of general international scientific societies to come to Italy for their conventions in the year just past, indicate that Italy has taken a vigorous new departure toward prosperity, largely based upon modern science.

DISCUSSION

C. McD. Townsend,* M. Am. Soc. C. E. (by letter).—The writer considers this paper the most valuable of the numerous productions on river hydraulics which have been evolved as a result of the flood of 1927 in the Mississippi Valley. While it contains numerous data which would be otherwise unavailable to most readers, its great value arises in that it recognizes that the science of river hydraulics existed in Italy prior to the discovery of America, and that Italian engineers have been engaged for centuries in investigating the laws governing the flow of water in streams.

While they may not have established laboratories satisfactory to the author, they have made practical experiments on the River Po for generations which are of much greater value. They have published and analyzed the results of their experiments, recording not only their successes but their failures. The description of the attempts to restore the river to the Grand Po after the break occurred near Ferrara, should be read carefully by those who would invite a similar catastrophe in the Mississippi River when constructing spillways.

The great value of the observations arises not only from the thoroughness with which they were made, but also from the long period of time which they embrace. German engineers have demonstrated that the construction of levees has not raised the bed of the Rhine below Lake Constance; those of France have arrived at a similar conclusion from an investigation of the flow of the Loire; and the Mississippi River Commission can find no trace of a river fill due to construction of levees on the Mississippi. The geologist, however, ignores these statements and continues to rely on M. Proney's report of the rising of the bed of the Po at Ferrara as recorded by Cuvier, notwithstanding the fact that it was denied and ridiculed by Italian engineers at the time it was made. It has also been claimed that the bed of the Yellow River has been raised due to levee construction. It is, therefore, a valuable addition to the science of river hydraulics when so great an authority as Mr. Freeman asserts, not only that the raising of the bed of the Po has been "found to be mostly untrue", but that the reports about the Yellow River "are mostly untrue, although not entirely so."

Another fallacy which has embarrassed river engineers is the assertion of Gustav Wex in a series of papers published from 1873 to 1879, that deforestation and drainage have caused a rise of the bed of the Danube. Schlichting and other German engineers analyzed the discharge of both the Rhine and Danube, and came to the conclusion that the little effect on the bed of these rivers which had occurred could be ascribed more logically to other causes. The masterly paper by G. Fantoli on "Il Po nelle Effermeridi di un Secolo" conclusively demonstrates that the only injurious results on the regimen of the Po caused by human agencies, have arisen from the diversion of water for irrigation; and that neither levee construction, deforestation, nor drainage has had an appreciable effect on its flow. Similar conclusions have also been derived from an analysis of the flow of the Upper Mississippi.

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* Col., U. S. A. (*Retired*), Washington, D. C.

Frisi and others decided that the troublesome elevations of river bed occur on the tributaries near where they leave the steep slopes of the narrow mountain ravines and enter upon the much more gentle slope across the main valley floor, and that gravel deposits are found in the upper reaches of the river, while in the lower sections the river bed is composed of finer material readily moved by moderate river currents. These deductions explain the cause of the errors committed by geologists, and indicate why they so readily accept, without investigation, any statement that the bed of a river is rising.

It is unquestionable that mountains and hills are eroded during rain storms and that the material thus moved is deposited in the valleys of streams or in the sea at their mouths. The geologist, however, fails to recognize that there exists, near the head-waters of every valley, a zone of deposition where the heavier portions of the eroded detritus are deposited, and that only the portions readily carried in suspension find their way to the sea. Succeeding floods again set in motion the material first deposited, but the grinding which ensues converts boulders into pebbles, pebbles into gravel, and gravel into sand, and in a relatively short distance gravel-bars are converted into sand-bars capable of being readily acted upon by river currents. This action occurs, not only on the tributaries of the Po, but on the Rhine above Lake Constance, and on the hill streams emptying into the Upper Mississippi. It is a rational explanation of the fill noted by the author on the Yellow River,* as this fill occurs below a point where the river debouches from a mountain gorge.

As the writer has explained in various pamphlets and statements, most of which have been interred in the *Congressional Record* and the proceedings of Congressional committees, the engineer in charge of the improvement of the navigation of rivers is only interested in that section of a stream which has been brought into unstable equilibrium by this grinding process. In this part of a river, the cause of the rise of the bed is its extension into the sea, as discussed by Mr. Freeman.

The writer has inspected both the Po and the larger rivers of France and is somewhat familiar with European literature on river hydraulics prior to the World War. He has had no opportunity to read the report of March 17, 1924, mentioned by the author, but he has studied the Atti del Comitato Technico Esecutivo, Commissione per la Navigazione Interna, decreto 14 Ottobre, 1903, in which, while suggesting the improvement of the Po by the French system of river control, the Commission recognizes the success of the dredging operations on the Mississippi River in maintaining a channel, and proposes further experiments with dredges before definitely adopting the project.

It is to be inferred from the statements made by the author, that the Italian engineers have finally adopted the report of October 14, 1903, after making experiments extending over a period of twenty-four years.

One of the United States Army Engineers recommended a similar project for the improvement of the Upper Mississippi River in 1898.† The improvement of the Missouri River is based on the same principle.‡

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* "Flood Problems in China", by John R. Freeman, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1422.

† *Journal*, Western Soc. of Engrs., Vol. XIV, p. 26.

‡ H. R. Doc. No. 1287, 61st Cong., 3d Sess.

The system of river improvement adopted for the Po has resulted from experiments made by M. Fargue on the Loire River in France and by M. Gerardin on the Rhine. It is the system which has received the approval of the Ecole des Ponts et Chaussées and has been fully described in the textbooks of that institution which have been published since 1899.

The assertion of the author that he would improve the project adopted by straightening the lower reaches of the river, indicates a failure to appreciate the principles governing this method of river regulation. He is proposing to substitute the German method of river regulation which the Italians (as he states) have abandoned, and is re-opening a discussion which has agitated river hydraulic engineers in Europe for many years, and, at present (1928), receives little support even in Germany.

Humphreys and Abbot demonstrated* that river-shortening led to a lowering of the low-water pools above a cut-off, and to an elevating of the river bed below. M. Gerardin was the first to recognize the injurious effects of such flattening of the slopes on the river's regimen, and to show the necessity of maintaining the alternation of pools and bars which Nature has created in a river.

Italian engineers appreciate that navigation would be irreparably ruined if the slopes on the lower river, which are now gentle, were still further reduced, and the slopes on the upper reaches, which are now excessive, increased thereby. They not only maintain the existing bars by giving a curved trace to their works, but they are especially careful to limit the river contraction so that they will not create a scour on the bars during flood stages. The limitations of the height of their works to 1 m. above low water is not primarily to reduce flood heights, but rather to invite a fill on bars during a rise in the river. When the river falls, the water is directed along predetermined lines so as to create a channel through the bar which will have the depth desired at all stages. It is for that reason that they have adopted the parabolic curve in their trace.

The extensive use of the waters of the Po for irrigation has rendered the problem more difficult than that occurring on the Upper Mississippi or on the Missouri. The curvature which the bank naturally assumes has, therefore, been more generally utilized in improving the rivers of the United States; and less care has been taken to give a prescribed slope to the contracting dikes.

It is noted that sand dikes and training walls used on the Po are approved by the author. Similar sand dikes have been extensively used by the U. S. Army Engineers in recent years on the Upper Mississippi, and have been equally successful on that river, although they are confined to the convex side of bends.

Permeable dikes were also tried experimentally on the Upper Mississippi more than forty years ago, but brush mattress dams were substituted finally because the river in its upper reaches carries little sediment in suspension even during floods. Below the mouth of the Missouri they have been eminently successful, although it was necessary to give them much greater strength than those described on the Po. In recent years, concrete piles have been quite generally substituted for those of wood, between the mouth of the Missouri River and Cairo, Ill.

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* "Report upon the Physics and Hydraulics of the Mississippi River," 1861.

The general dimensions of the levees on the River Po were known to the Mississippi River Commission before the existing levee section for that river was adopted, and the sections developed by the Germans on the Rhine and by the French on the Loire were also studied. European levees have generally a greater width of crown and less width at the base than the Mississippi River levees, due principally to the fact that European levees are used extensively as roadways.

Formerly, the usual road of the Southern States consisted of undrained soil without ballast which was nearly impassable during rainy weather. It would have been the height of folly to have permitted the tops of levees to be converted into a similar quagmire by the passage of vehicles over them. It was recognized moreover that the ultimate height of levees had not been definitely determined, and that, if macadam roads were constructed at the low grades then existing, they would have to be rebuilt from time to time. The wisdom of the decision is forcibly illustrated by the results of the flood of 1927. Every road which might have been in existence on the tops of levees would now require reconstruction.

The Commission therefore decided to give to the levee such a form as would offer the maximum resistance against water pressure, with the most economical movement of the earth which formed it. This incidentally created a banquette on which a road could be located whenever the progress in road building reached the stage that the road surface was protected from abrasion. The minimum requirements were a width of crown of 8 ft., and a width of base equal to ten times the probable head of water against the levee. These dimensions were not based on laboratory tests, but on observations of the resistance of the average soil of the Mississippi Valley to filtration. In many localities, not only in the valley of the Mississippi, but also in the valley of the Po, soil is found which is so porous that levees of the dimensions here given are unsafe, but when the muck ditch reveals such a soil, the levee engineers simply enlarge the section by flattening the slopes.

From the writer's inspection of the Po Valley, he gained the impression that the second line of levees were irrigation ditches in which water is carried at considerable heights above the general surface of the ground. He would not, however, question a statement by an authority as competent as Senator Luiggi. He suggests nevertheless that a second line at the same elevation would exceed in cost that of one along the river bank, since the ground of an alluvial valley slopes away from the river. The financial condition of Italy hardly justifies such a luxury.

If the space between the two levee lines is allowed to fill during a rise of the river, it will materially reduce flood heights. Whatever may be the attitude of the farmers of Lombardy, the writer is convinced that the planters along the Mississippi River would never consent to such a proceeding. There are double lines of levees along certain sections of the Rhine, but they conform to the conditions shown on Fig. 10, a front line overflowed at medium stages and a controlling line to regulate extreme floods. The writer has seen a statement in a German publication that this peculiar construction was adopted as a compromise measure to satisfy owners of vineyards who had a theory that the restriction of floods to the front line of levees would have an injurious effect on the grape crop of the adjoining hills.

Science would be greatly benefited if the Treasury of the United States could afford the expenditure and Congress would authorize the establishment of rain gauges throughout this country at as frequent intervals as in the Po Valley. Such data would be extremely valuable for determining the volume of water which causes a flood, when time has been afforded to assemble and digest them; but as a means of flood prediction they would be practically useless. A flood would have passed before a central office could have computed the percentage of flow that would reach the river. The method adopted by the U. S. Weather Bureau for determining flood heights by gauge readings on the tributaries is far more practicable and also more accurate, because it measures the run-off and avoids the necessity of computing the quantity of water evaporated and absorbed by the soil. The flood predictions of the Weather Bureau compare most favorably with those of European nations.

A marked advance in flood prediction was also made by U. S. Army Engineers during the flood of 1927 in the Mississippi River. The determination of the height that the water would attain in the area flooded by crevasses, was of inestimable value to Secretary Hoover in his efforts to rescue flood sufferers, particularly the work of Capt. Lewis A. Pick in the valleys of the Teche and Atchafalaya Rivers.

The observations at Pontelagoscuro accurately determine the quantity of material carried in suspension by the River Po at different stages. They however merely supplement the work of the Mississippi River Commission and of U. S. Army Engineers in the investigation of the flow of sediment in streams. The observations on the Mississippi River not only trace the flow of material in suspension from its source in the tributaries to its deposition in the Gulf of Mexico, but also measure the quantity of material moving along the river bed in sand waves. Furthermore, they indicate the effect of slope, curvature, and discharge on the height and permanency of sand-bars, and the relation of caving banks to turbidity.

The author invites attention to the employment of civil instead of military engineers in improving the rivers and harbors of Italy, but he fails to recognize that the Italian civil engineers are taught the fundamental principles of the science of river hydraulics in the various engineering schools which he enumerates, while the Italian Army Engineer does not have such instruction. In the United States, conditions are reversed. The U. S. Army Engineer is taught at least the rudiments of the science at the Army School at Fort Humphreys, while it is very generally ignored in the various engineering colleges of the country. The assistant engineers who have been employed and frequently have been instructed by the Army Officer are the only other body of engineers who have had practical experience with the problems which arise in the improvement of rivers and harbors in the United States.

The author will be of great service to American engineers if he succeeds in his efforts to persuade Italian engineers to translate into English, their knowledge of river hydraulics. With such information available, it is even possible that it will be unnecessary to establish a laboratory in Washington for the purpose of rediscovering principles known in Italy for centuries and now taught in the ordinary textbooks of the engineering schools of Europe.

Oren Reed,* Assoc. M. Am. Soc. C. E. (by letter).—With the 1927 flood on the Mississippi River as a reminder, American hydraulic engineers will welcome the author's excellent description of the flood problems of the River Po in Italy. For centuries various methods have been tried in an effort to control the flood flow of the Po, with only partial success. As their knowledge of river behavior has become more complete, the Italian engineers have been able to lessen the damage from floods. Certain practices, which have been proved by long experience to be wise, could be adopted with profit by American engineers.

One point which was not mentioned by the author is the influence of lakes, either natural or artificial, in reducing flood flow. As is shown in Fig. 1, the south slope of the Alps receives the largest precipitation of the Po water-shed. This drainage is partly regulated by large lakes—Garda, Iseo, Como, Lugano, and Maggiore. The regulating effect of these lakes would be very great without any artificial control. Lake Maggiore has an area of about 83 sq. miles. A record of its maximum and minimum water levels shows that the yearly variation is from 6 to 16 ft.† This large annual variation would indicate that in a wet year the lake had temporarily retained a large volume of water which would materially affect the flood discharge of the Lower Po.

The effect of natural retention in lakes was clearly illustrated in September, 1927, by Lake Constance, which is located on the German-Swiss border. The head-waters of the Rhine River in Switzerland were swollen by a severe two-day storm in the last days of September, 1927. The quantity of water carried by the Rhine above Lake Constance on September 25 and 26, 1927, was the greatest yet observed. The flow into Lake Constance from the Rhine on September 25 reached a maximum of 81 300 sec-ft. The total inflow into the lake was 148 500 sec-ft. The maximum outflow occurred on September 29, and was only 29 700 sec-ft.‡

In Norway, investigations have recently been completed to establish a comprehensive plan for flood regulation on the Skien River. This study was prompted by a flood of high intensity in June, 1927, which caused great property loss. The communities affected by the high water, appointed committees for the purpose of perfecting a plan for preventing a similar catastrophe in the future. The technical study was under the direction of Christian Raestad.

The Skien is the third largest water-shed in Norway and has a drainage area of 4 060 sq. miles at the City of Skien, which is about six miles from the North Sea. Nearly all the area is mountainous and includes numerous lakes and fjords. The head-waters flow from a plateau region, a large part of which is marsh land.

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* Care, San Joaquin Light & Power Corporation, Fresno, Calif.

† *Schweizerische Wasserwirtschaft*, February, 1928, p. 21.

‡ *Loc. cit.*, p. 17.

The Skien District is one of the most important in Norway. It has long been noted as a timber-producing district. "Norsk Hydro" (the Norwegian Electric Company) is located in the central part of the water-shed about 65 miles from the ocean. This Company, the leading industrial concern in Norway, uses hydro-electric power for the fixation of atmospheric nitrogen. The total plant capacity controlled by Norsk Hydro amounts to about 260 000 kw.

Dams have been constructed at the outlets of a number of large lakes in the Skien water-shed for the purpose of regulating the flow for power and for the floating of timber. Most of these regulated lakes are in the upper part of the water-shed. The most important ones are: Mos Lake, 597 000 acre-ft.; Maar Lake, 183 000 acre-ft.; and Tinn Lake, 159 000 acre-ft.

Authentic records of floods on the Skien River begin about 1645. Serious floods have occurred at frequent intervals, but none has caused as much property damage as the flood of 1927. The banks of the river had been encroached upon in recent years by industries and private interests. After the regulation of Mos and Maar Lakes the low-lying areas were built upon, for it was expected that flood levels would be reduced. The 1927 flood was caused by heavy precipitation, augmented by melting snow, on the lower part of the water-shed. Regulation of the upper lakes decreased the possible flood, because no water was released from Mos Lake during the flood period.

Floods on the Skien have usually occurred in the late spring and are caused, as in 1927, by heavy precipitation and melting snow. There have been, however, many autumn floods. The highest recorded flood at the City of Skien was in June, 1927, and amounted to about 106 000 sec-ft., or 26.2 sec-ft. per sq. mile of total area.

The recommendations of the Committee appointed in 1927 were: (*a*) To increase the capacity of the stream at critical points; and (*b*) to decrease the flood flow by further regulation of natural lakes.

At the City of Skien and at Skot Fall, crib dams serve for diversion to low-head power plants. During flood periods these dams materially raise the water level. It is proposed to remove these crib dams and construct new structures, regulating the flow by means of roller or sector gates. During floods, the gates would be opened and would present no obstruction to the flow. At certain points considerable dredging will be necessary to obtain sufficient channel capacity for the maximum possible flood.

The flow in the upper part of the drainage basin is regulated at present in Mos, Maar, and Tinn Lakes. Additional regulation is planned for the middle section of the water-shed at Lake Totak and at Sundsbarn. It is thought that the maximum probable flood can be economically decreased about 21 000 sec-ft. by the proper regulation of these lakes. This would be about 20% of the 1927 flood.

By regulation more of the flow can be made useful for industries and power plants. This will be a valuable feature, although the chief purpose of the work will be a reduction of flood flow. It is thought that the work can be financed by assessments on the communities and industries benefited without any aid from the State. For reasons of economy, it will be necessary to do the work in two or more steps.

John P. Hogan,* M. Am. Soc. C. E.—The author has called attention to the fact that there really is a flood-control proposition that is being studied in a scientific manner. This is particularly appropriate at a time when, during the past year or year and one-half (1927-28) engineers have been attempting to handle a flood-control proposition partly on a political basis and partly on a sentimental basis. The speaker refers, of course, to the Mississippi River.

It would seem that in order to provide flood protection, the first and foremost thing would be the necessity of determining what the maximum flood might be. Allen Hazen, M. Am. Soc. C. E., has given a very great contribution to this subject† which, of course, is as applicable to flood control as it is to river regulation. Nevertheless, the law of probabilities will never foretell the maximum flood that will occur. It will only predict the probability of the occurrence of a flood. It may be known that a certain flood will occur once in 100 years, once in 20 years, or once in 10 years, but it will not be possible to predict that perhaps to-morrow, or next year, there is going to be the flood which is due once in 10 000 years. It would seem to be axiomatic that absolute and positive flood protection by completely confining the waters of a river within a narrow channel is not economically feasible, and there is considerable doubt as to whether it is practically possible.

Without any further study and with very indefinite knowledge on the subject, efforts have been made to pass a blanket resolution providing indefinite sums of money to control the Mississippi River. There has been absolutely no effort whatever made to accompany the plans for control with any economic study, to determine whether such a regulation is advisable. It certainly is evident that it does not pay to spend $1 000 000 000 to save $100 000 000.

It would also seem evident that the river must have a channel with certain marginal districts of land subject to periodic flooding, as Mr. Freeman has shown in regard to the Arno. It would seem that the first process in determining a reasonable plan of regulation for the Mississippi River would be an estimate of the present value of the land affected and its prospective value. A determination could be made, on that basis, of the present and future methods that should be used either to reclaim or preserve from flood at the present time the maximum acreage of land which might economically be reclaimed, and to provide for certain future measures that could be applied if and when the reclaimed land becomes sufficiently valuable to warrant it. Further large expenditures might be justified on the broad grounds of public welfare, but the speaker believes that the economic survey should be the first step.

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* (Parsons, Klapp, Brinckerhoff and Douglas), New York, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1539.

Land owners have been crowding into the lowlands for the past 50 or 100 years as land became more valuable. They have been filling up the channels which were formerly open to the river in time of flood. It would seem that it is about time, as far as a river like the Mississippi is concerned, to take stock, and to find what should be reclaimed and preserved either in the present or in the future.

In presenting this view of flood control on the River Po in Italy, Mr. Freeman has done a real service.

William B. Gregory,* M. Am. Soc. C. E.—The speaker is very much pleased with Mr. Freeman's paper. It is a splendid example of the ways in which hydraulic information should be gathered and applied to a problem such as that of the Po.

There are many people in New Orleans, La., who think because they have lived on the banks of the Mississippi River and have seen it all their lives, and because their fathers saw it all their lives, that they know all about it; and one of the most common fallacies is the belief that the bed of the river is rising. They have seen the levees rise as the years go by; in fact, in the thirty years that the speaker has lived in New Orleans the levees in the front of the city have been raised about 6 ft.; and, of course, people who are not acquainted with hydraulic laws jump to the conclusion that the bed of the river is rising, which is not true. What Mr. Freeman has emphasized in connection with the Po, is also true of the Mississippi; namely, that it is not scouring out a deeper bed, as was hoped years ago.

The Mississippi River Commission began with the idea that confining the river within levees would scour out a deeper channel and improve that channel. In an address at Memphis, Tenn., C. McD. Townsend, M. Am. Soc. C. E., stated that while that might be true extending over a long period of time, it would be of more interest to the people who will occupy the valley in the Twenty-fifth Century than to those who live there now.

Incidentally, Colonel Townsend has stated† that the Mississippi River Commission had every bit of information that is necessary for the control of that river, and all that needs to be done for it in the future.

John R. Slattery,‡ M. Am. Soc. C. E.—It seems always of great importance to bring out the fact that the bed of the Po and the bed of the Mississippi have not risen. At least, if the bed of the Mississippi has risen, it has been at so slow a rate that as far as the engineers of the present day are concerned it need not be considered. That misleading observation of the old priest traveling along the Po has caused more sorrow and grief to engineers engaged in flood control on alluvial streams than almost any other fallacy.

A few years ago, C. McD. Townsend, M. Am. Soc. C. E., made a most exhaustive study of the Italian records. He did not read or speak the language before that time, but he studied enough Italian to delve into the records of the Po River, and his paper on that subject was one of the most conclusive that has been written. The paper was not published; it is in the archives of the Mississippi River Commission.

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* Prof. of Experimental Eng., Tulane Univ., New Orleans, La.

† *The Military Engineer*, March–April, 1928, p. 93.

‡ Deputy Chf. Engr., Board of Transportation of the City of New York, New York, N. Y.

Spur-dikes or retards have been tried on many rivers in this country as well as in Europe. The great trouble with that system of improvement seems to be that, while something is accomplished, the end sought is never fully realized. A system of spur-dikes to produce a channel 9 ft. deep may be designed. The system is completed, some improvement results, but the desired 9-ft. channel is rarely, if ever, obtained. An elaborate system of dikes of that character has been under construction above Cairo, Ill., on the Mississippi River, for a number of years and has produced a marked improvement.

In early years some attempt was made at Lake Providence, or in the Providence Reach and the Plum Point Reach, at river control, and it was more or less successful. It was abandoned because of the great expense and because through the development of dredging it was unquestionably cheaper to maintain the necessary depth by this method than by a system of permanent structures.

The flood-fighting methods on the Po, mentioned by Mr. Freeman, are quite similar to those used on the Mississippi. In fact, those officers who have been engaged on the Mississippi are made to study quite thoroughly the history of the Po as well as of all other similar rivers. Many plans are thought to be new when as a matter of fact they have been tried and failed. The Mississippi River Commission has been liberal in providing money for the investigation and trial of really new ideas, but has properly set its face against wasting money in investigation of plans already thoroughly tried out without satisfactory results.

One of the great difficulties on the Mississippi has been the divided control. In the 1916 flood the speaker saw the gauge at Cairo gradually rising, and about three weeks before the crest reached Vicksburg, Miss., it was realized that the flood that year would be the highest that had ever been known. It was evident at that time that a severe fight was on hand. It was evident, also, that certain stretches of levees would have to be built up with sand bags.

As the flood advanced, the necessity of carefully guarding the levee was realized. Most levee problems (if the levee is not overtopped), can be controlled, provided warning is received in sufficient time and that the proper supply measures are taken at the beginning. The speaker, therefore, through the newspapers of Greenville, Vicksburg, and the small towns that published papers, urged all owners having plantations facing on the levee system to undertake the patroling of their own levees.

They all have large numbers of negroes; there is no scarcity of help. These people are practically carried through the winter season anyway by the planters for whom they work. There was immediate and violent opposition on the part of one levee official because of what he seemed to think was the usurpation of some of his authority, and it took a great deal of diplomacy before he was finally mollified and brought to co-operate in securing the proper guarding of that levee system.

Millions of sacks that year were purchased, 6 000 000 in that one district, and distributed to various warehouses along the river. The plant of the district consisted of 5 towboats and about 80 barges. Those were distributed at strategic points along the river. Barges loaded with gravel and lumber were stored in the lower reaches where the back-water up the Yazoo Valley attains almost the same height on the levee as the water on the outside.

The levee line was carefully watched; a warning was sent of a boil at this point; a sloughing bank at that point; water lapping over the levee at another point; and within a few hours it was possible to move men and supplies to any point on that river, and that was the way that the levee system was held. At three o'clock one morning a steamboat captain telephoned that the water was running over the levee a little north of Vicksburg for a distance of about 5 miles. At seven o'clock the next morning barges of gravel and sand bags were at that particular point. The back-water was so high that no material could be obtained from the levee itself. Nevertheless, if the levee were lost, it meant an extension of the flood a number of miles up the river. The actual difference between the level of the back-water and the water in the river at that point was about 5 ft. It took a week of fighting, but at the end of the first day there was a line of sacks above the flood, and that flood was followed up until it crested by adding line after line of sand bags.

Some of the methods that are now suggested for the improvement of the river have received most careful consideration in former years. One reason why they have never been adopted has been the strong local opposition to anything but levees.

The Cypress Creek Gap was closed while the speaker was in Vicksburg, or rather, it was started. It was realized that the closing of that gap would immediately raise the height of floods for some distance below the mouth of the Arkansas, because the closure cut off an enormous reservoir in Arkansas and Louisiana. Much of the land in this region was not particularly valuable; a great deal of it was not and probably is not to-day under cultivation; a great deal of it was wooded; there were many cotton plantations so infested with boll weevil that cultivation had ceased. At the same time the local pressure to stop the overflow, which took place at a stage of 49 ft. at Arkansas City, was so great that the Mississippi Commission had to close the gap. The 1922 flood confirmed, to within 1 ft., calculations made in 1916, as to the effect, of closing that gap, on river stages at Arkansas City and vicinity.

The overflows are not always an unmitigated curse. No planter wants to see his land overflowed; but many a planter, after a crevasse has occurred, finds that the following year the increased fertility of the soil more than offsets his loss for the one year of flood. It sometimes happens that, instead of coating his plantation with a film of rich silt, sand is carried in and his plantation is ruined for years to come, so it is a gamble. There are parts of the Mississippi Valley to-day that would be greatly benefited if they could be overflowed. The soil has been worn out, and the rich plantations that once existed there cannot now pay expenses.

The Po and the Mississippi are near relatives, but the levee section on the Mississippi is quite different. There has long been consideration given to the desirability of putting a road on these levees. The river slopes on the Mississippi levees are 1 on 3, or, in certain soils, 1 on 4. The top width is 8 ft. The slope down to the banquette on the land side is 1 on 3. The banquette is from 8 to 12 ft. below the crest, and varies in width from 20 to 40 ft., with a slope of 1 on 6. The lower slope on the land side is about 1 on 4. This heavier section is necessary, as Mr. Freeman pointed out, because of the long period that the water stays high; and on the Mississippi those fighting the floods do not feel safe until the water has fallen about 10 ft. below the crest of the levees. Even after the water starts to fall, there is still danger of these levees failing with the reduced head through sloughing.

On the matter of cut-offs, or the straightening of rivers, in this country the general rule is not to straighten a river except where streams, such as those in Florida, have a very slight slope. There, the streams are materially shortened to their great advantage. On the Mississippi, however, every effort is made to prevent cut-offs. Above Greenville, for instance, there is a long bend around a long point of land. It was 5 miles around this point and only about 2 400 ft. across it. A cut-off would have shortened the river about 4 miles.

The river began to cut channels across the point, and an attempt was made to fill them by permeable dikes. They were not successful. It was impossible to induce those channels to fill. The banks were heavily revetted along the upper side of the point, but some signs of failure were noted. It was recognized that if the neck was cut through, at least one-half the levee system for perhaps 50 miles below would go as a result of the bank caving that would follow from the excessive currents that would exist until such time as the river re-adjusted its slope.

An earthen dike about 3 or 4 miles long was built from the levee system out to near the point. Straightening the Mississippi is not to be thought of unless the banks and bottom for many miles above and below cut-offs can be adequately protected against scour. To any one familiar with conditions on the Mississippi above Baton Rouge, La., the problem of preparing the bed to carry the river safely at a greater slope, comes within the realm of economical impossibilities.

ARTHUR P. DAVIS,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—The River Po presents unusual opportunities for the study of river hydraulics, not only on account of its opportunities of physical conditions, but also due to the long history of its use and modification by human agencies. Mr. Freeman's paper is of great interest and shows industrious research into the history and literature of this river.

The typical large river system, of which many examples can be found in the world, is a collection of drainage from high or mountainous regions, which flows to the sea, eroding its channels in the upper reaches and depositing the eroded material at its mouth. In the course of hundreds of thousands, or perhaps millions, of years, the basin is lowered some hundreds or thousands of feet by the influence of the waters of this river system. Where the climate is humid and the rainfall considerable, the hilltops are weathered and disintegrated by the action of the rain and frost, and the tendency is to produce a rounded topography such as that seen in the Appalachian Mountains, and in the main in the Rocky Mountain System as well. If the river through its course carries its load of water and sediment through an arid region where the surface weathering is slight and where the declivity of the river permits rapid erosion, deep and narrow canyons may be produced, as in the Colorado System of the Southwest.

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* Chf. Cons. Engr., Sredazrodhoz, Tashkent, Turkestan, Union of Socialistic Soviet Republics.

Throughout the history of such a river system, the forces of Nature are actively at work degrading the topography toward the condition known by geologists as "base level", and, if not interfered with, the entire basin with lapse of time, would be reduced to a "base level" or a region so flat that any movement of water thereon would be so sluggish as to produce no erosion. Limited areas of such base levels occur near the mouth of every large deltaic stream, such as Southern Louisiana on the Mississippi, and the great delta of peat land area near the junction of the Sacramento and San Joaquin Rivers in California.

Whether the river has declivity enough to produce erosive velocities, it continues to erode its channel until this is reduced to an elevation such that the fall to the sea is insufficient to produce such velocity. The slope that is necessary varies, of course, with the quantity of water flowing, the larger quantities producing higher velocities, so that there is a critical stage in some part of such rivers where the channel has a tendency to erode at high stages and to refill at low stages.

The work of the river through the centuries, bringing down its load of sediment, builds a larger and larger delta at its mouth through which the stream meanders a sluggish course at ordinary stages, depositing the load that it has brought from the mountains. Many interesting examples of this may be cited.

The Yangtze-Kiang in China has advanced its mouth thirty miles into the Yellow Sea by progressive delta building during historic times. The great deltas of the Mississippi and Sacramento Rivers are well known in detail. The Colorado built a delta at its mouth extending from the California line entirely across the Gulf of California, severing the head of the Gulf from the main body and forming thereby an inland sea which evaporated and formed the Imperial Valley lying below sea level, with a strongly saline lake in the bottom. This delta of the Colorado River has been built to an elevation of 40 ft. above sea level at its summit and reaches north and south nearly 100 miles.

As the mouth of such a river advances into the sea, the increased length causes a reduction of the gradient of the stream which is emphasized also by the reduction of the elevations higher up by erosion. This decrease of gradient deprives the river of part of its carrying power and causes the deposit of a portion of its sediment farther and farther up stream; so that for some distance up stream from its mouth there is a gradual tendency to the upbuilding of its channel. As this upbuilding progresses, the river bed rises and creates a tendency to overflow which is characteristic of every such river throughout its lower course. As the river begins to overflow during the flood, the water overflowing the banks is checked in velocity where it encounters the obstruction of vegetation and shallow depths, and immediately deposits the coarser materials it is carrying. This tends to build up the immediate banks of the river, while the partly clarified water spreads out over the valley and deposits a thinner layer of sediment. Thus is formed the characteristic topography of a river channel as it approaches the base level condition, where the banks of the river are higher than the grounds immediately back of them and there is a tendency for the river to run on a ridge as illustrated in the paper (see Fig. 13).

This unstable condition cannot continue, however, and some time at flood the river breaks the banks and flows through the lower country in a new channel where, in time, the same process is repeated; and thus the river swings back and forth in the course of centuries and builds up a broad alluvial valley to an extent sufficient to maintain such velocities as will carry a portion of its sediment to the sea. Thus, the combination of conditions involves the processes, (*a*) the degrading influence throughout the mountainous regions of the stream near its head-waters; (*b*) the erosion of its channel through the upper course; (*c*) the building of a broad alluvial valley through its lower course; and (*d*) the extension of the delta at its mouth into the sea.

The writer visited the Po Valley in 1911 and examined its largest irrigation system, and by these observations and the impressions produced by the literature on the subject, concluded that the Po River was typical of the conditions described. It is probably one of the best types to be found, rising as it does in the lofty Alps and Apennines with their high precipitation and heavy slopes; it has wide valleys with deep alluvial soil throughout its lower courses, and a characteristically large delta at its multiple mouths.

As the Po River still comes from the high Alps and Apennines, affording abundant precipitation and abundant gradient, erosion of the channels through the mountainous regions must still be in active progress, and this material must be deposited either at the mouth of the river, or in the valley, or both.

It is interesting therefore to find that, in the "Introduction", the author makes the following statement:

> "Flood heights certainly have increased, but the best data available show that meanwhile the bed of the main channel has not been raised."

This statement is doubtless based on the report which the author quotes from the Ministry of Public Works, as follows:

> "As seen from the said studies of the Hydrographic Office of the Po and from the hydrometric observations of the Po for a century, it is evident that the bed of the river presents no tendency to a progressive rising."

This conclusion, in view of the surrounding facts, is interesting, for if no sediment is deposited in the river bed, it must all be carried to its mouth and deposited in the Adriatic where it would lengthen the course of the river and progressively reduce its gradient.

The confinement of the river between dikes would prevent the deposit of sediment over the floor of the valley and confine it either to the river bed or to the delta in the Adriatic and would actually accelerate the growth of the delta and the consequent rise of the river bed. This tendency could be postponed and even for a time it could be reversed by straightening the river channel, which would reduce its effective length and thereby increase its gradient; and this, in turn, would increase its velocity and prevent deposits in the river bed. If such were the case, the eroded material from the mountains would travel to the sea and the growth of the delta would be accelerated and, in time, would add to the length of the river sufficiently to counteract the shortening produced by straightening. This seems to be the most plausible explanation of the apparent anomaly already noted. Mr. Freeman's opinion on this explanation would be interesting and valuable.

If this artificial straightening has not occurred, what then is the explanation of the fact that the building up of the river bed, which has been proceeding "during hundreds of thousands of years by the sand and mud eroded from the mountain slopes", has recently stopped?

Whether the bed of the Po, or of the Mississippi, or of the Yellow River, or of any other deltaic stream, has been built to an elevation higher than the adjacent country, is a question only of speed and of quantity. Hydrologists should not lose sight of the fact that all such rivers are eroding material from their upper reaches, bringing it down into the lower valleys where it is all consumed in enlarging its delta, lengthening its course, or building up its valley and river bed, or both.

If the river is confined so that it cannot deposit its sediment in the broad valley, all this sediment must then be utilized in building up the river bed or extending the delta, or both. In the long run both must occur, else what is to become of this sediment?

The Rio Grande and some other streams have varied this program temporarily due to the operations of Man. In the case of the Rio Grande, the head-waters were consumed largely in the irrigation of upper valleys and the steady flow of considerable quantities of water which formerly occurred have been largely stopped. This same flow consists mostly of clear water resulting from melting snow, and the reduction of its volume and the shortening of the season of its discharge by consumption above have deprived the river of a large proportion of its carrying capacity through the middle and lower reaches. The result is that the torrential tributaries which bring their loads of sand and gravel to the valley during sudden storms, build up their deltas faster than the main stream can carry them away and the reduction of the summer flow has greatly accelerated this process. Some parts of the bed of the Rio Grande have risen during the past fifty years from 2 to 12 ft. by actual levels, which is doubtless much faster than the rate of rise before the water was diverted at various points.

The most complete plan for the control of the floods of the Lower Colorado involves not only the regulation of the river through large storage works, but the straightening of its channel in the lower course for a distance of about 100 miles above its mouth. It is believed that it will be practicable to shorten this stretch of the river 30 or 40% by confining it between straight dikes set as close together as conditions permit and preventing the river from meandering. This, of course, will add greatly to the effective gradient of this section of the river and will cause it, for a long time to come, to erode its channel and thus remove the tendency to overflow. This effect, however, can be only temporary and will eventually be overcome by the deposits in the delta, the lengthening of the channel, and the loss of gradient resulting therefrom.

Such operations, of course, would result in a condition in which the river in this section would present "no tendency to a progressive rising", as reported by the Hydrographic Office of the Po. This would not set aside the general principle that these alluvial streams have been for thousands of years, and will be in the future, progressively building up their channels and their valleys and extending their deltas with the sediment brought down from the mountains; nor the further fact that if the stream is prevented by dikes from depositing this sediment over the wide valley, it will greatly increase the amount that must be devoted to extending the delta and building up the river channel, unless the sediment is stored above, which is not the case on the Po.

These tendencies may be modified by the storage of sediment in reservoirs near the head-waters; by the diversion of water for irrigation; and by the straightening of the channels; or, perhaps, by some other operations of Man. The effect of these operations is necessarily temporary, geologically speaking, and eventually the time must come when the river will resume building up its bed.

It would be interesting to know just what operations of Man have modified these natural tendencies in the Po Valley, and how permanent they may be.

G. de Thierry,* Esq. (by letter).—This paper gives an excellent picture of the development of the regulation of a river on the banks of which the cradle of hydraulics has stood.

On the Po, as on all other rivers, the first works erected by the hand of Man were for the purpose of protection against the dangers incident to high water. The attempt to protect as much of the adjoining land as possible from floods has led, on the Po as well as elsewhere, to a narrowing of the river channel which doubtless has constituted in places an obstacle to the free run-off of high water. The attempt to remove these obstacles subsequently has met with opposition which has been the stronger, the further the utilization of the land behind the levees has progressed. Such levees can naturally occasion a rise in high-water levels, so that frequently erroneous conclusions regarding the raising of the river bed are drawn from this phenomenon. Particularly valuable is the clear statement that neither on the Po nor on the Yellow River in China can any raising of the river bed be traced to the effect of levees.

There is only a single particularly characteristic case of the raising of the bed of a river known to the writer. For decades it has been known that on the section of the Rhine above the Bodensee, opposite the Principality of Lichtenstein, a rise in the river bed amounting to 2 cm. yearly exists. The Rhine is diked on both sides along this section; but it would be a serious error to attribute the rise in the river bed to these parallel levees. The building of control structures along the mountain streams above this section of the Rhine is still far behind what it should be because of the very considerable cost involved. Consequently, after heavy rainfalls, or a sudden melting of snow, these mountain streams bring down enormous quantities of gravel which are in continual motion and which the river cannot digest. It is quite possible that the effect of the cut-offs and the new outlet into the Bodensee will extend far enough up stream to prevent the deposit of this gravel. In any event it will be a long time before the effect in this part of the river will be demonstrable. The creation of a new outlet into the Baltic Sea has produced a far-reaching reduction in the level of the highest water on the lower reaches of the Vistula River. The increase in the velocity involved in the shortening of the stream has brought with it a lowering of the river bed, so that on the Rhine above the Bodensee a similar effect is to be expected.

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* Berlin, Germany.

However, the Rhine below Basel, Switzerland, has proved that a long period of time must elapse before a verdict can be given. A century ago, at the instigation of Oberst Tulla, numerous cut-offs were undertaken between Basel and the Hesse-Prussian border, reducing the length of the river to 81 km. Enormous quantities of gravel were thereby set in motion. On the upper part of this stretch of river the deepening of the bed exceeded 3 m.; farther down stream, the bed was raised. Although this progressive change in the river bed dates back fully a century, it is by no means finished, and it is scarcely possible to predict when it will come to an end. However, in this case again, the cause is not to be traced back to levees.

During the writer's visit to the United States in 1927, he observed repeatedly that the Army engineers considered that there was no object in building a hydraulic laboratory. They contend that the accumulated results of experience gained by working with Nature itself are more valuable than the study of natural processes in the laboratory. They thus overlook the fact that most of the processes in the realm of hydraulics are of an extremely complicated nature, and that the observation of particular phenomena in Nature permits only in the rarest instances an analysis of the individual factors which have operated together to produce the phenomena observed. The investigation of cause and effect is the ultimate goal of science, and the hydraulic engineers in Germany are convinced, after 30 years' experience, that the hydraulic laboratory is an indispensable aid in the development of this science.

River problems are among the most difficult that the hydraulic laboratory is called upon to solve. The river in its natural state is the result of numerous factors, not only of a meteorological, but also of a geological, nature. The processes of flow, the distribution of the flow during the year—not only of the quantity of water corresponding to each stage, but also the duration of a particular flow, which fluctuates from year to year within certain limits—affects the course of the river. Only in the rarest cases is the problem confined to the same kind of geological formation in a river valley. It is quite possible to reproduce a stretch of river which has been accurately examined geologically and to investigate on this the effect of particular structures; but when the results of laboratory tests are to be carried over to Nature, one must always expect that, through accidents, through changes in the nature of the river bed, through snags or sunken vessels which are often invisible to the engineer, effects are produced which evade all attempts to reproduce them in a model test. Certainly, indications as to the best solution of a particular problem can be obtained through the laboratory, but there is little hope that, with the erection of one or more laboratories, the difficult problem of the Mississippi can be completely solved.

When, at the end of the Nineteenth Century, devastating floods occurred on various German rivers, there was appointed, at the command of the Kaiser, a commission whose function was to investigate the causes of the different flood catastrophes. The results of this investigation, which was conducted with unusual thoroughness, are compiled in a series of works which comprise an imposing library.

Mr. Freeman deals with the question of whether it is more practical to entrust the work to be carried out on the Mississippi to engineers in civil life in place of the existing military organization. This is a matter which must be settled within the United States itself.

The writer believes, however, that the author has made a very important suggestion, the application of which should not be disregarded. The Po, like every other river, is, in effect, a living organism, the study of which requires more than the lifetime of any one man. First, through systematic work, all the data can be brought together to give a unified picture of the organism which the engineer is to attempt to improve. Every interruption in the continuity of this scientific work is injurious. The unified and complete picture which Mr. Freeman has given makes it obvious that in more than a century of systematic work the fundamental facts for the investigation of all the factors affecting the formation of the river have been collected and assimilated.

M. GIANDOTTI,* ESQ. (by letter).—This paper describes very well the actual conditions—geo-physical, morphological, and hydraulic—that occur on the River Po.

The tributaries of the Po may be considered as divided into two distinct classes (depending on their origin), namely, Alpine and Apennine. The most important Alpine tributaries are the Ticino, Adda, Oglio, and Mincio Rivers, which flow through Lakes Maggiore, Como, Iseo, and Garda, respectively. This common characteristic of these rivers to form lakes provides a natural settling basin, and the tributaries thus deposit all sediment carried in suspension at these places. Below the lakes the rivers are quite clear and carry only some fine sand and mud, so that the Po receives no sediment from these rivers. The clear water tends, rather, to clean out the bed of the river, even removing some of the sediment brought down from the Apennines. The water-sheds that drain into the Apennine tributaries are almost entirely composed of calcites and argillaceous materials and, therefore, the streams are very turbid. In a manner similar to the Alpine tributaries, the Apennine rivers deposit the heavier material in so-called "expansion beds" which extend up stream from the Bologna-Milan Highway for 50 or 60 km. These beds have a variable width of from 1 km. to about 100 m. and a slope not greater than 5 to 8 per cent. After the heavier material has settled out, the rivers

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* Chf. Insp., Civ. Eng. Dept., Po River, Parma, Italy.

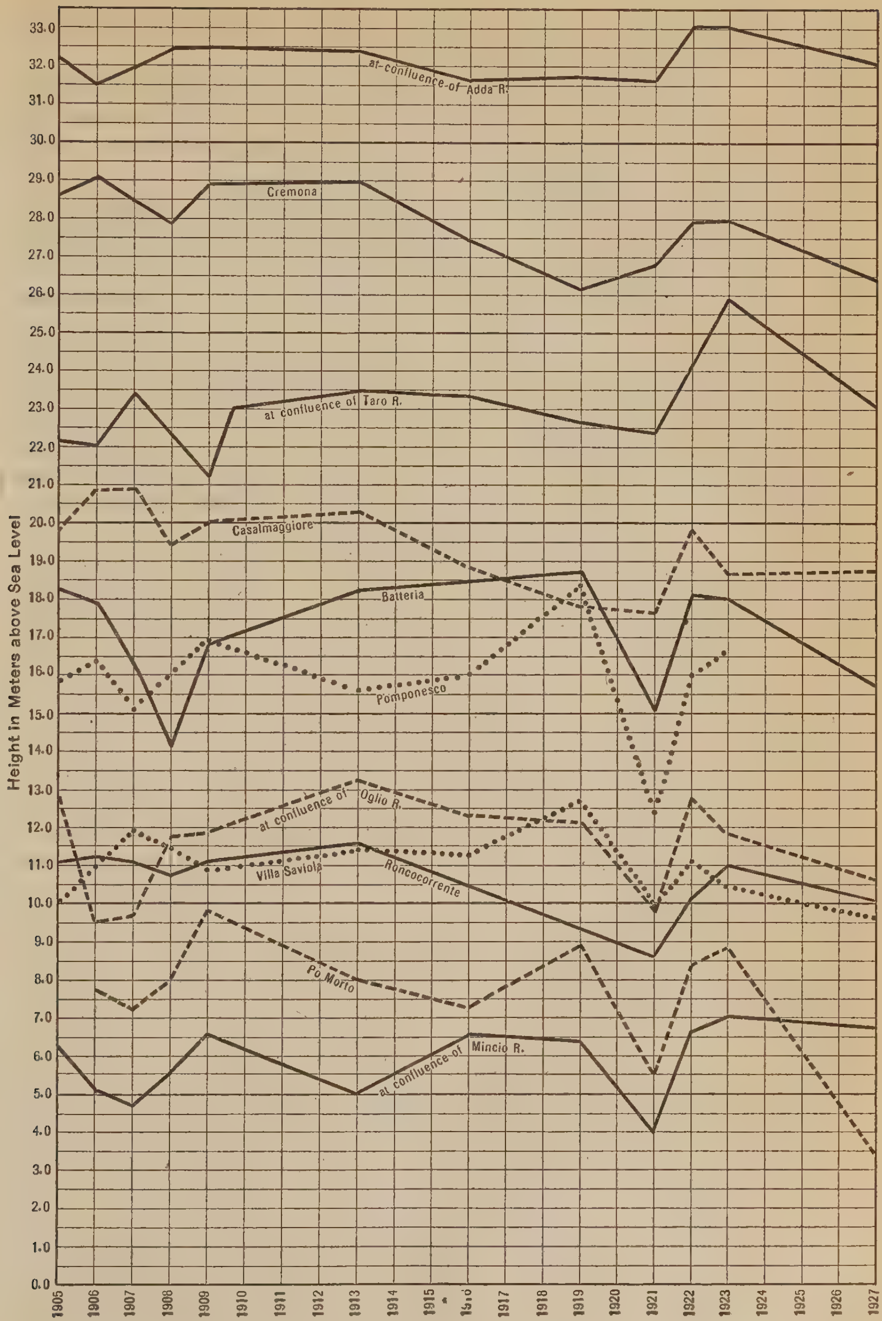

FIG. 14.—CHANGES IN MEAN ELEVATION OF BED OF RIVER PO AT GAUGING STATIONS FROM 1905 TO 1927.

flow in eroded beds with a sand bottom and hence carry nothing to the Po except fine materials which are easily carried away by floods, or, as previously stated, by the clear waters of the Alpine tributaries.

There is no sign that indicates, historically, a noticeable advance of the gravels and none that marks any appreciable rise in the river beds, because the floods that occur in these rivers are of short duration and are limited in general to the spring and autumn seasons of the year.

At any rate, the gravel that settles in the expansion beds is heavy enough so that the present slopes may be considerably increased before any of it will be picked up and carried into the eroded river beds below. In the meantime, Italian engineers are providing against such a contingency by reforestation, etc.

This is true also for that part of the Po from the mouth of the Adda River (at Cremona) to the outlet at the sea. In the section from Turin to Cremona several torrents like the Trebbia, the Scrivia, Tanaro, Sesia, Dora Riparia, Dora Baltea, and the Orco Rivers transport gravel to the Po. In the section extending down to Ticino, the bed of the main river is entirely gravel, and the river flow is of a torrential character. The remedy consists in controlling the torrents that flow into the Po and regulating their delta cones.

The slope of the valley on either side of the Po is only 1 or 2% near Cremona. Farther down the river the water level, during ordinary floods, is nearly always above the surrounding country. In this region the adjacent lands are drained by pumping or by means of separate canals that empty directly into the sea.

Lands adjacent to some of the tributaries, such as the Taro, Parma, Enza, Secchia, and Panaro Rivers, are drained by means of pumps.

There is not a second line of dikes along the Po. The Government's dikes form a continuous line of defense down to the sea. However, the lands that are included between these dikes and the lower course of the river are protected by private or communal secondary dikes. These are all lower than the main dikes by at least a meter and, therefore, they are from 20 to 30 cm. below the maximum flood.

In regard to the permanence of the bed of the River Po, Fig. 14 shows the mean elevation of the bed observed at gauging stations for a period of years. The readings are taken at the same place each year. While they are never the same from year to year, the elevations always oscillate about a line of equilibrium and show practically no appreciable net change.

This permanence of the average river depth is also clearly demonstrated by other hydraulic phenomena* and experimental results, as follows:

(*a*) A comparison of cross-sections taken during 1916 and 1917 with those observed by the Brioschi Commission, about 1878, show that the area of discharge has not changed. In certain cases, it has increased.

(*b*) The average decennial scalar diagrams of the minimum annual depths at Pontelagoscuro, revised to date by Professor Fantoli,† show an undulating course with alternate minima and maxima, and do not indicate lowering or raising the level of the river bottom.

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* Relazione ed Allegati Ministero dei Lavori Publici, Parma, 1924, p. 84.

† "The Po in the Ephemerides of a Century."

R. D. Goodrich,* M. Am. Soc. C. E. (by letter).—The author has given the profession a very valuable résumé of the history and development of the regulation of the River Po. To those who have become interested in this particular branch of engineering, the records of the observations, investigations, and experience of the eminent engineers who have been engaged on this work in Italy, are not of passing interest only, but are invaluable as sources of information and reference. Hence the paper is of unusual interest and is especially timely on account of the situation connected with the Mississippi River.

While the science of river engineering doubtless had its birth in Italy some centuries ago, the art has been practiced in some of the countries of the Far East for more than 2 000 years. Nowhere is this more true than in the Provinces of Shantung and Chihli, in North China. Some comparisons and contrasts of the practice and conditions there with those in Italy, as described in the paper, will be of value.

One of the largest rivers of the Province of Chihli is the Yung Ting Ho which presents one of the most difficult and interesting problems of regulation and flood control in North China. The single exception is the Yellow River which is, of course, a much larger and longer stream with many unique conditions introducing many special problems.

In 1918 a commission, of somewhat international character, was organized by the Chinese Government to carry out some much needed flood-protection and regulation works on the rivers of Chihli, and to survey the general situation and recommend measures for flood relief and the control of its rivers. This Commission therefore has been studying this river, with others, for a short time, although it has been more or less under human control for at least 500 years.

It is about 80 miles from the foothills just below the canyon by which the Yung Ting Ho leaves the mountains, to the confluence with the Hai Ho at Tientsin, 30 miles from the present sea coast. Practically no drainage enters the river below the mountains as this portion has been controlled for a long time by a system of dikes. Above the lowest gorge on the river, the effective drainage area is about 18 000 sq. miles. The basin is approximately 200 miles long and about one-half as wide, although the shape is irregular. The average annual precipitation at Tientsin, near the coast, is about 20 in., and at Peking, just below the mountains, it is 24 in. However, on the plateaus and mountains comprising the drainage area of this river, the rainfall is very much less than on the coastal plains; but the data are not sufficient for an accurate estimate of the average for the basin as a whole. For this river, therefore, the general conditions as to rainfall on the coastal plains and mountain areas are exactly the reverse of those on the Po, while the amount is probably only about one-third that in Northern Italy.

Through the gorge, the river has a slope of about 1 : 300. Above, it meanders across the Huai Lai Plain for a distance of about 15 miles on a slope of about 1 : 1 000. This plain is at an elevation of about 720 ft. above sea level. To the north and west, above this plain, the river drains a series of mountain ranges and high plateaus.

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* Prof., Civ. Eng., Coll. of Eng., Univ. of Wyoming, Laramie, Wyo.

Summer is the flood season in North China, with the heaviest storms between July 10 and August 20. However, the disastrous effects of the worst storms may not occur near the coast until the waters which accumulate behind the levees on the plains reach their maximum height. Under present conditions this may not happen until late in September, as in 1917. These great floods, which inundate thousands of square miles of agricultural land, are caused by a succession of typhonic storms. That of July 15 to 17, 1924, produced a maximum discharge of about 176 000 cu. ft. per sec. below the foothills, or about seven-tenths the maximum rate for the Po, in 1917. This is an average run-off for the Yung Ting Ho of almost 10 sec-ft. per sq. mile for the effective drainage area, which may be compared with the 15 sec-ft. stated for the Italian stream. Mr. Freeman's "possible" explanation for the relatively small run-off per square mile, would seem to apply with even greater force to the Yung Ting Ho where such a large part of the drainage basin is of loess formation, and where there is never any frost in the ground at the time of such storms. The total run-off from this basin for the months of July and August, 1924, was estimated at 53 300 000 000 cu. ft., while the precipitation was approximately 459 000 000 000 cu. ft. This is a run-off of about 11½% for the two months of the flood season.

While these comparisons show that the Po is the larger stream with greater unit run-off and maximum discharge, the discharge of silt by the Yung Ting Ho is much greater. This is perhaps more remarkable than any other feature of these floods of North China, and particularly so for this river. Samples have been taken which showed 10% of silt by weight for the Yung Ting Ho and as high as 11% for the Yellow River. These are of course maximum records, and it is quite probable that such high degrees of saturation are not usually of long duration. Hence, they would have relatively smaller effect on the total weight of silt transported annually; but amounts of from 2 to 5% by weight are not at all uncommon for much longer periods during the flood season. At the same time conditions often arise which tend toward high degrees of saturation locally; for example, when there is rapid erosion of high banks or loess cliffs, which cave off into swift currents so that the stream carries a very heavy load of silt for a short distance. The writer saw a whole village destroyed in three days in 1924 when the Pei Ho was in flood. Deep scouring undercut the banks so rapidly and the shifting of the channel was so inevitable, that the villagers were powerless to prevent the disaster. In fact, the channel has shifted about a mile from east to west within three years at this point.

Such conditions, where the river has built up the plain mostly of this loess formation, and where enormous quantities of similar material are transported annually, probably present the most important factors tending to cause the river to meander by distinct and widely separated channels in this region. The migrations of the Yellow River have been mapped and discussed at various times, and the Yung Ting Ho has had a very similar life history during almost 2 000 years of record.

The average annual volume of silt deposited by the Yung Ting Ho on its way from the mountains to the sea, has been estimated at 13 000 000 cu. yd., in addition to which an unknown volume has been carried on and added to the yellow waters of the Gulf of Chihli to be spread finally on the bed of the sea or along the constantly extended shore. Approximately, this would be 14 000 000 metric tons of material used annually in building up the flood-plain and bed of this river. While the data on which this estimate is based are not conclusive, this subject has been given very careful study and the figures are believed to be conservative. This mass of silt is more than double that brought down by the Po with its larger basin, and yet, be it remembered, does not include that carried to sea. If this material were distributed uniformly over the entire effective drainage area of this river, it would amount to about 850 short tons per sq. mile, which is moved annually from the upper reaches of the river to the plains between the mountains and the sea. While the total amount is certainly much greater than that stated, it should also be remembered that the great bulk of this material comes from certain well-defined areas forming enormous deposits of loess. Much might perhaps be accomplished if this silt problem were attacked at its source, but as yet little or nothing has been done.

For a few miles below the mouth of the gorge where the river leaves the mountains, the bed is composed of very coarse gravel. Less than ten miles from the foot of the mountains the bed is composed almost entirely of sand. Farther down, this material becomes finer and finer, until there is nothing but silt in evidence on the more level plains. Unlike most rivers, the Yung Ting Ho does not now have any well-defined delta cone at the point where it leaves the mountains, due to the fact that it has been trained for centuries between well-maintained dikes, some having been built about 1350 A. D. Although this river now flows into the Pei Ho at a point 30 or 40 miles from the sea, it does have a "delta", so-called from the manner of its formation. This "delta" is from 6 to 20 ft. above the level of the surrounding plain. At its head are at least three distinct lines of dikes constructed to confine the river as it built up its flood-plain and bed, and this building is still going on, as it has for 200 years. Contrary to the opinion of the Italian authority, Frisi, the point where the building up of the bed is most rapid, is 40 or 50 miles below the last sign of gravel in the bed of the stream. Where the river has built up its bed well above the plain the sand and silt forming the river channel is extremely porous, causing enormous seepage losses. In June, 1924, a month before one of the highest floods of recent years, the river was dry at the head of the delta.

It is evident that with a reduction of 100% in the volume of flow, all silt must be deposited, and with a reduction of 50%, which is not uncommon, the amount deposited must be very large, with a decided tendency to maintain the degree of saturation, if it does not actually increase as one proceeds down stream. If the mouth of the Yung Ting Ho is taken at its junction with the Pei Ho on the east side of the delta, the change in elevation at this point in all probability has been little more than it would have been were it building out the coast line at its mouth for the same length of time. The size of this river, the volume and character of the silt transported, the topography and geological formation of its drainage basin, as well as the climatic conditions, are all so different from those obtaining on the Po, that different results and conclusions would seem to be almost inevitable.

In addition to the dikes along the Yung Ting Ho, other devices have been used to assist in the control and regulation of floods. The simplest structure is an open spillway, or notch, in one of the dikes, with abutments and wing-walls which, as a rule, are built of excellent ashlar masonry laid with lime mortar. The crests and aprons are usually a mixture of earth and lime, which forms a sort of concrete. Under certain conditions, and with properly selected materials and good workmanship, this method of construction gives satisfactory results. In other instances abutments and wings as well as the aprons and crest, are of earth-lime concrete, and have stood for almost 200 years. A temporary dike is usually built on the crest of a spillway to a lower elevation than the main dike so that it will be over-topped or can be cut in case of an excessive flood.

Another spillway of cut-stone masonry, is in excellent condition after nearly 200 years of service. It was intended to be controlled by stop-logs, but the crest elevation is such that only the higher floods overflow it and at present (1928) no attempt is made at regulation.

Still another spillway was built for better control of floods on the Yung Ting Ho, as late as 1914, but this one shows foreign (probably French) influence in design and construction. It is located at the west end of the so-called Marco Polo Bridge near Peking, a most interesting and very ancient structure mentioned by that noted traveler. Between these last-mentioned weirs, the river dikes are very wide apart and form a channel reservoir having an area of more than 20 sq. miles, which has great effect in reducing flood peaks on account of the relatively large storage effect and also on account of the considerable absorption which takes place.

The earth dikes have piles of material distributed along the top for emergency repairs. This is common practice although the earth available is usually not sufficient for more than very minor repairs. The elevation of the channel reservoir above the surrounding country, together with the very porous nature of the deposits made by the river along its flood channel, doubtless accounts for the extraordinary seepage losses along this reach of the river. It is estimated that for a flood peak of 5 000 cu. m. above Lu Kou Chiao, about 1 100 cu. m. will be discharged over the weir at that place, while about 450 cu. m. more will flow over the cut-stone spillway. Between these points there is a probable further reduction in the crest discharge of about 700 cu. m. for the reasons stated.

All this may serve to emphasize the well-known facts that, while observation and experience with one stream are valuable in that the statement and analysis of the problems involved on that stream may be recorded for comparison, each river presents its own peculiar problems due to its own peculiar conditions. Only by the most careful study of the conditions and problems for each individual case, can one hope to arrive at anything approaching a satisfactory solution.

C. E. Grunsky,* Past-President, Am. Soc. C. E. (by letter).—Current engineering literature is filled with accounts of works and structures recently completed and with discussions of pending problems. It is refreshing to get from time to time what Mr. Freeman has given the profession—an account of works which have functioned for centuries and which experience has shown to be adapted to the purpose for which they were constructed.

In the matter of flood control, the Engineering Profession has much to learn from this experience. There is only one point in connection therewith which, it seems to the writer, should be further stressed. The valley of the Po differs from the valleys of the Sacramento and the Mississippi mainly in the fact that the principal flood-control levees can be set far back from the stream in the upper parts of the valley. Broad areas of cultivable land protected against moderately high river stages by low submersible levees—summer dikes—flank both sides of the river. These have the same purpose, and function in the same way at extreme river stages as the by-pass areas in the Sacramento Valley. In the latter case, however, the areas set apart for occasional flooding could not, for topographical reasons, be located along the river; they had to be set back some distance from the high bank land. In both cases, however, the purpose is the same, that is, large cross-sectional area is provided in or along the river's upper reaches to hold water in temporary storage, thereby cutting down the maximum rate of discharge at points in the valley farther down stream.

John R. Freeman,† Past-President, Am. Soc. C. E. (by letter).—The kindly words of Colonel Townsend about the information given in the paper are most welcome, especially his hearty appreciation of the work of the early Italian engineers. These men were the founders of hydraulic science, notwithstanding the fact that it required more than a century of their earnest work to place the square root sign in the formula for the discharge from an orifice, and that a hundred years still later it remained for a French engineer to invent the Chezy formula for stream flow.

During the Middle Ages, Northern Italy was the world's greatest center of universities, and those who governed the country called into service for river regulation and flood protection the foremost professors of physics and mathematics of their time. The writer believes that students in the new schools of hydraulic research at the great engineering colleges of Italy—with the aid of recently established hydraulic laboratories, and the new observations on the River Po—will learn more of controlling these waters economically in the next forty years than has been learned by work on the river alone in the past four hundred years.

Colonel Townsend refers to the fact that the Italian civil engineers are taught the fundamental principles of river hydraulics in the various engineering colleges of that country, while this subject is commonly ignored in American engineering colleges. The writer understands that almost every important college in Europe has a Professorship in River and Harbor Hydraulics, while there is not yet one in any college on the American Continent.

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† Cons. Hydr. Engr., Providence, R. I.

The fundamental reason for this is that no attractive career opens, even to the brightest student from the foremost engineering college—whichever that might be—if he begins and continues as a United States Assistant Engineer. He can never rise to the top and have his name attached to any important structure (*vide* the experience of the late Alfred Noble, Past-President, Am. Soc. C. E.). Up to the present time the only door opening to such a career is by way of the National Military Academy at West Point. While the eminent Army engineer may retire at the age of 65, with a pension of about $5 000 per annum, and thereafter carry on a lucrative consulting practice, even the ablest civilian engineer on river and harbor work in the United States must either resign, or work on at about one-half the salary of the West Point graduate until he is 70 years of age, when he will be dismissed, with a pension of only about $1 000 per annum.

The Army method, which largely makes voucher clerks of the Resident Army Engineer's staff, and after a very few years on any one problem shifts the leading officers to new fields, cannot possibly produce the best results for research on river and harbor improvement; but it is doubtless the best peace-time method of training Army officers for a future war. Army methods assume that any one man can replace any other man of equal grade—and that is a necessity in the field of battle; but the fundamental facts of human nature remain—that probably not one good average engineer out of ten (perhaps only one out of a hundred) has the inborn talent to become a great research man and develop that initiative which advances the science and the art.

Colonel Townsend deprecates the need of hydraulic laboratories for studying river problems. If he should visit Italy and Germany again he would be amazed, as the writer was, to observe the development of that most useful of aids in experimental research, the Doctrine of Similitude, with the use of small, relatively inexpensive models, which is now placed in the hands of the skilled engineer striving to build better hydraulic turbines, and of those engaged in river and harbor engineering. It is the strongest tool ever invented for advancing the state of the hydraulic arts.

In due course it may revolutionize methods in research on river flow, and tidal action in harbors, just as the naval tank in the hands of Froude, fifty years ago, gave a new departure for problems in ship modeling.

So rapid and recent is this progress in the method of attack on hydraulic problems* that many people have overlooked the remarkable research made in the hydraulic laboratory of the Engineering College at Manchester, England, on the projected creation of a great tidal water power by means of a dam across the estuary of the Severn River. It is interesting to note that a model of the river and bay, 40 miles long, is housed in a room 45 ft. square, and is constructed with such nicety and accuracy of detail that it cost about $10 000.

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* "Hydraulic Laboratory Practice," pub. January 1, 1929, by the Am. Soc. Mech. Engrs., New York, N. Y.

The laboratory idea is developing and spreading so fast in Europe that, within the last year or two, great new hydraulic laboratories have been brought to completion by the Swiss Government at Zurich; by the Netherlands Government at Delft; the Norwegian Government at Trondhjem; and the Czecho-Slovakian Government at Prague. Surely such hydraulic laboratories are not built, as Colonel Townsend suggests, "for the purpose of rediscovering principles known in Italy for centuries and now taught in the ordinary textbooks of the engineering schools of Europe." It is the firm belief of the writer that tens of millions of dollars have been wasted on river and harbor work in the United States through failure to realize the value of the hydraulic laboratory and the applications of the doctrine of hydraulic similitude.

It is difficult for a stranger in a foreign land unfamiliar with the language to make sure that he correctly understands the data placed before him. Hence the writer values the discussion of Professor Giandotti particularly because of its acknowledgment that the "paper describes very well the actual conditions". Professor Giandotti emphasizes some of the finer points that the writer missed. Furthermore, the discussion is valuable because of the proof given in Fig. 14 that, although the elevations of the river bed fluctuate from year to year, as the volume of sediment is moved along, and that "while they are never the same from year to year, the elevations always oscillate about a line of equilibrium and show practically no appreciable net change."

The discussion by Professor Goodrich is of interest because of comparisons of conditions along the Po with those along certain waters in Chihli in Northern China. This confirms similar statements once made before the Society by the writer which at first were regarded as unbelievable. The percentage in volume of silt to water brought down by the River Po is surely very moderate as compared with that of some of these Chinese rivers.

Mr. Grunsky's contribution is of particular interest in further emphasizing the difference in the problems of flood control presented by rivers in various parts of the United States and of the world. Surely, however, the fundamental laws and principles of erosion, sedimentation, and stream flow, apply to all. The differences mentioned by Mr. Grunsky result simply from different values of the factors in the same general equation, presented by the variable circumstances of the drainage area and the precipitation in the several localities. The writer has been impressed with the differences in sediments, or materials, shoved along by the current, in rivers with rapid slope leading out from the Alps as compared with those found in the beds of most American rivers in the East. On the other hand, in rivers coming down the western slopes of the Sierras and the Cascade Mountains gravels and cobblestones are carried along as in the Alpine rivers. The mechanical laws are of universal application.

Regarding Colonel Slattery's dictum, that it is "unquestionably cheaper" to maintain the necessary depth by dredging than by spur-dikes and retards, the writer suggests that dredging was long ago developed to the maximum possibilities of efficiency, while spur-dikes and retards of twenty years ago may have fallen very far short of the best and most economical that could be devised by further studies in the laboratory and in the field. The time has come to re-open some of the questions that were considered "settled" forty years ago.

Colonel Slattery well describes the marvelous efficiency with which levees were topped out by sand-bags during the Mississippi flood of 1916. The writer, as a student of river hydraulics, followed the greater flood of 1922 along down the river, and was thrilled by the wonderful rapidity and efficiency with which levees were pieced up, sand-bags placed, and threatened crevasses checked; and he fully believes that no better or more energetic work has been accomplished anywhere in the world, in times of such great emergency. Much of it was done by civilian engineers and chiefs of levee districts. Nevertheless, the work along the Po in the great flood of 1927, by which 30 miles of sand-bag dikes were placed on top of the dikes of the river, which rises much more rapidly than the Mississippi, is of a similar order of excellence and probably superior in some of the details. The sudden rise of the Po permits no such relatively leisurely preparation in collecting grain sacks from a thousand miles away as is possible along the Lower Mississippi, where the flood height following abnormal rains can be predicted with a fair degree of precision several weeks in advance.

Regarding the fact, that "overflows [along the Mississippi] are not always an unmitigated curse", the writer believes that something can be learned from the system along the River Po, where there are two sets of dikes, one near the river and a higher one farther back, with numerous cross-sections, thus permitting an occasional overflow and enrichment of the broad strip near the river. So far as the writer could learn, the valley floor of land subject to inundation along the Po does not pitch back from the river, or decline in altitude, to any such degree as that along the Mississippi, where, in general, the bottom-lands near the river are fully 5 ft. higher than they are a mile back. This condition along the Mississippi is adverse to arranging a system of double dikes as along the Po; but the writer is strongly inclined to believe that thorough study would show that a double dike system in certain parts of the Mississippi delta would give the best possible protection, and by proper design of intakes would permit periodically flooding the intermediate land for restoration of fertility. In contrast to the Po, the dike nearest the Mississippi should be the main line of defense, and the series of secondary dikes, running in straight lines about ½ mile back from the apex of principal bends, should here be regarded as the second line of defense, which would prevent the flood from escaping and flooding, for example, the whole Yazoo Valley from an unsuspected sand-boil and midnight crevasses.

Replying to the questions of Mr. Davis, the writer found that there was apparently no doubt among Italian engineers that the Po was steadily lengthening its delta, and projecting its load of sediment into the Adriatic, thereby constantly lengthening the river's course.

Dr. de Thierry, who was born in Northern Italy, knows the history, the language, and the work of the Italian engineers. Perhaps his most useful contribution to this discussion is that about the time factor involved in the study and working out of these great river problems, which he suggests is a matter of a century or so; and, that "the Po, like every other river, is, in effect, a living organism, the study of which requires more than the lifetime of any one man."

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Paper No. 1727

ONE HUNDRED FIFTY YEARS ADVANCE IN STRUCTURAL ANALYSIS*

By H. M. Westergaard,† M. Am. Soc. C. E.

With Discussion by Messrs. S. Timoshenko, Alfred D. Flinn, Jacob Feld, and H. M. Westergaard.

Synopsis

The story of the advance of structural analysis is, in the main, an account of the work done by individual men. While the science of structural analysis is international, at the same time its national traditions are significant. In the French tradition the names of Coulomb, Navier, and Saint-Venant stand out; in the English tradition the name of Rankine; in the German, the names of Mohr, Müller-Breslau, and Föppl; and there is a Russian tradition which may be traced back to the influence of Euler.

The account which follows begins, somewhat arbitrarily, but (as will be seen) not entirely without justification, with the work of Coulomb. Consideration is given, however, to his predecessors. An attempt is made to sketch the development from this early period to the present.

The ancient bridges, the vaults, arches, and flying buttresses of the great medieval cathedrals, and the riggings of sailing vessels at the height of their development, bear witness of structural insight on the part of the builders; but the art of structural analysis, except for small beginnings, is only about 150 years old.

Structural analysis is based on the physical properties of materials; its method is mathematical; its purpose is design. It came into existence by a meeting of physics, mathematics, and engineering.

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* Presented at the meeting of the Structural Division, Philadelphia, Pa., October 8, 1926.

† Prof. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

COULOMB, 1776

In 1776, the Paris Academy of Sciences published a paper by Charles Augustin Coulomb, entitled, "Essai sur une application des règles de Maximis et Minimis à quelques Problèmes de Statique, relatifs à l'Architecture".* Coulomb, who lived from 1736 to 1806, was a military engineer. He possessed mathematical insight and attained distinction as a physicist in the fields of mechanics, electricity, and magnetism. Discussing beams with rectangular cross-section, in the paper of 1776, he assumed the fibers to offer resistance proportional to their extension or shortening; he considered the balancing of the internal forces acting upon a cross-section; and he determined, by accurate reasoning, the position of the neutral axis and the moment of the internal forces. He remarked that at rupture, in some cases, the neutral axis may be at a different position. In the same paper, he considered the deformation by shear in connection with the failure of a solid; and he presented his theory of earth pressure on a retaining wall, according to which a wedge of earth slides when the friction and cohesion in a plane section become insufficient.

Saint-Venant remarked eighty years later† that in this paper "one finds presented almost all the bases of the theory of stability of structures." If one considers the combined circumstances of Coulomb's capacity for exact science and his early relation to practical affairs of engineering, it is not strange that he should be the author of so significant a document.

It is perhaps stretching a point to state that, because of this paper, Coulomb is the "Father" of structural analysis. He certainly is one of the "Fathers".

COULOMB'S PREDECESSORS

It would be unreasonable, however, not to consider Coulomb's predecessors.‡ Galileo, in a dialogue published in 1638, inquired into the problem of the strength of a cantilever. He considered the beam to be perfectly rigid except for the turning about a horizontal axis in the section of rupture. In spite of certain incorrect assumptions he obtained a correct design of a cantilever of uniform strength for the case of a rectangular cross-section. Hooke, in 1678, announced his famous law that deformations are proportional to the loads. Mariotte at about the same time arrived at the same law, and applied it to the fibers of a beam. Tests (made by him in 1680) led him to the observation that some of the fibers of a beam are stretched and others shortened. He placed arbitrarily the boundary between extension and shortening at the middle of the beam. Varignon, to whom credit belongs for early developments in statics§ (for example, the polygon of equilibrium of a string), discussed the investigations of Galileo and Mariotte.‖ Varignon computed a moment of resistance of a cantilever by assuming the tensile force of the fibers to be proportional to the distance from an axis, but, unfortunately, in spite of Mariotte's observation of shortened fibers, he placed this axis at the bottom of the cross-section.

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* In the volume for 1773 of "Mémoires de Mathématique et de Physique, Présentés à l'Académie Royale des Sciences, par divers Savans, et lus dans ses Assemblées," pp. 343-382.

† *Journal de Mathématiques Pures et Appliquées*, 2. Série, v. 1, 1856, p. 90.

‡ A very complete account of the development of ideas and methods relating to the theory of elasticity and strength of materials may be found in the great work of Todhunter and Pearson, "A History of the Theory of Elasticity and of the Strength of Materials," Cambridge Univ. Press, 1886–93. A shorter account is given by A. E. H. Love in the historical introduction to his "Mathematical Theory of Elasticity," Third Edition, 1920. See, also, H. Lorenz, "Technische Elastizitätslehre," 1913, pp. 644-683.

§ P. Varignon, "Nouvelle mécanique ou statique," 2 vol., Paris, 1725.

‖ In a paper published in the *Mémoires* of the Paris Academy, in 1702.

James Bernouilli, the earliest member of the distinguished Bernouilli family to attain fame as a mathematician, at the end of the Seventeenth and the beginning of the Eighteenth Centuries introduced the problem of determining the shape of the elastic curve. After him is named the assumption that a plane cross-section of a beam remains plane after bending. He arrived at the peculiar, inadmissible conclusion that the position of the neutral axis is indifferent. His nephew, Daniel Bernouilli, about 1735, obtained a differential equation for transverse vibrations of a bar.

Leonhard Euler was born in 1707 in Basel, Switzerland, the city of the Bernouillis. In 1727 he was called to the St. Petersburg Academy by Catherine I, and in 1741 to the Berlin Academy by Frederick the Great. He returned to St. Petersburg in 1766 by invitation of Catherine II, and died in 1783. He was endowed with a magnificent intuition, and was one of the great mathematicians of all times. The Bernouillis influenced him to attack the question of the elastic curve. In order to understand his method, it should be noted that the science of physics in the Eighteenth Century was still struggling to emancipate itself from metaphysics. In a manner characteristic of this state of development, he argued* from the premise of the perfection of the universe to a principle of least action. In this way he established a principle of minimum for the flexure of beams, and he gave solutions for a number of cases. In a later paper, of 1757, entitled "Sur la force de colonnes",† he derived the celebrated formula which is named after him, and which expresses the critical load at which a slender column buckles.

In this early work on the elastic curve of beams and columns the quantity which is now represented by the product of the modulus of elasticity and the moment of inertia was given as a single "moment of stiffness", characteristic of the beam or column. Euler suggested determining experimentally this moment of stiffness ("moment du ressort" or "moment de roideur") by supporting the beam or column as a cantilever and measuring the deflection at the end, due to a transverse force at that point, a case in which the deflection could be obtained by a simple formula.

In addition to his work on the static elastic curves, Euler, during his later years, investigated successfully the transverse vibrations of beams, and, by an attempt to analyze the vibrations of bells, he began the work on the two-dimensional problems of plates and shells.

Progress and Relapses, 1776-1820

Consider again the paper by Coulomb, published in 1776. In spite of the unquestioned importance of the work of his predecessors, especially that of the great mathematician, Euler, Coulomb's paper stands out as a most important original document produced by an engineer, that deals with structural analysis for the sake of the structure. This paper does contain, in addition to other important material, the first adequate analysis of the fiber stresses in a beam.

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* Euler, "Methodus inveniendi lineas curvas maximi minimive proprietate gaudentes," Lausanne, 1744, Additamentum I: "De curvis elasticis."

† Published in the *Mémoires* de l'Académie de Berlin, v. 13, 1759.

It must not be thought, however, that Coulomb's analysis of the beam at once settled this issue in the minds of all others who were concerned with it. Girard published, in 1798, a treatise on the resistance of solids.* A German translation was published in 1803. This book is the first treatise on mechanics of materials. It has a historical introduction, and contains extensive accounts of the earlier works from Galileo to Euler and Lagrange. It reports tests performed on wooden beams by the author, and it is significant on account of the importance attached to experiments. Tests are in, and metaphysics is out. Girard, however, accepted James Bernouilli's incorrect conclusion that the position of the neutral axis is indifferent.

The learned Dr. Thomas Young, famous as a physicist and a philologist, made two lasting contributions to the subject of elasticity.† One is the discovery of the detrusion, or shearing deformation, as an elastic deformation (Coulomb had considered it as a permanent set in connection with failure); the other is the introduction of the modulus of elasticity, which is often named after him. Young, still, was uncertain and obscure as to the position of the neutral axis.

The textbooks from the beginning of the Nineteenth Century, in dealing with the subject of the strength of beams, are not very satisfactory. Olinthus Gregory, in his "Treatise of Mechanics, Theoretical, Practical, and Descriptive", London, 1806 (and in the later edition, 1815), accepted Galileo's results, because they are simple, and ignored the others. Todhunter and Pearson‡ refer to this book as evidence of "the depth to which English mechanical knowledge had sunk at the commencement of the nineteenth century." The book by John Banks, "On the Power of Machines" (1803), contains practical rules and erratic theory for the design of beams. Eytelwein,§ who possessed better mathematical understanding than his two contemporaries just mentioned, introduced the subject of beams well, but fell into the old error of placing the neutral axis in the concave surface of the beam.

In France, Duleau in his "Essai théorique et experimental sur la résistance du fer forgé" (Paris, 1820), quoted Coulomb on the subject of the neutral axis, but misunderstood him. In England, Tredgold in his two standard textbooks, "The Elementary Principles of Carpentry" (London, 1820), and "A Practical Essay on the Strength of Cast Iron" (London, 1822), displayed confusion in matters of theory. Hodgkinson, an able experimenter, contributed toward correcting the current errors about the neutral axis, especially through two papers published in 1824 and 1831 by the Manchester Literary and Philosophical Society. Barlow was erratic on the subject. In the edition of 1837 of his "Treatise on the Strength of Timber and Cast Iron,

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* P. S. Girard, "Traité analytique de la résistance des solides, et des solides d'égale résistance, auquel on a joint une suite de nouvelles expériences sur la force, et l'élasticité spécifique des bois de chêne et de sapin," Paris, 1798.

† "Course of Lectures on Natural Philosophy and the Mechanical Arts," Lond., 1807.

‡ "A History of the Theory of Elasticity and of the Strength of Materials," 1886–93. v. 1, p. 88

§ "Handbuch der Statik fester Körper," Berlin, 1808.

Malleable Iron and Other Materials" (page 63), he finally adopted Coulomb's principle for determining the neutral axis. He credited this principle to Hodgkinson.

THE FOUNDING OF THE THEORY OF ELASTICITY, 1820-1830

The picture of advance in structural analysis again becomes cheerful when one turns the attention toward France during the decade from 1820 to 1830. Three names stand out: Navier (1785-1836), Cauchy (1789-1857), and Poisson (1781-1840). These three men are considered to be the founders of the theory of elasticity. Navier, who became Professor of Mathematical Analysis and Mechanics at the École Polytechnique in Paris, combined ability in mathematics with competence in practical affairs of engineering. Cauchy entered the École Polytechnique in 1805 and the École des Ponts et Chaussées in 1807, and began a career as an engineer. In 1813, however, the two great mathematicians, Lagrange and Laplace, influenced him to renounce engineering for mathematics. Poisson, like Navier, was a Professor at the École Polytechnique. He was an ingenious and successful mathematician. His interests were directed toward pure science rather than toward its applications.

It was in 1821 that Navier submitted to the Paris Academy the investigation by which he originated the theory of elasticity of three-dimensional solids. By considering forces acting between the molecules according to a definite law of attraction and repulsion he established, for the first time, a set of equations for the equilibrium and the vibrations of the interior parts of a solid. At about the same time interest in the questions of waves of light aroused Cauchy to a study of the mechanics of an elastic medium. He succeeded, before the end of 1822, in setting up the fundamental relations of elasticity (in the form now current), in terms of the principal directions of stresses and strains, the six components of stresses and strains, respectively, and in terms of differential equations for the three components of displacement. Cauchy, later, added studies of the elasticity of crystals, which, on account of different properties in different directions, may have as many as twenty-one independent elastic constants (that is, constants of the nature of Young's modulus or Poisson's ratio). Poisson's most important contribution was presented to the Paris Academy in 1828*. This paper, besides fundamental theory, contains numerous specific applications.

It should be remarked that the theory of elasticity is primarily physics, aimed at the understanding of matter. The development of the fundamental processes of theory, through the past one hundred years, has been the joint work of physicists, mathematicians, and engineers. Applications to the molecular theory and the theory of sound have presented themselves. At the same time, applications to structural analysis have been a cause of continual contact with engineering. These practical applications to engineering have come into the foreground during more recent years.

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* Published in the *Mémoires*, v. 8, 1829, pp. 357–570.

MECHANICS OF ENGINEERING ACCORDING TO NAVIER

Navier, scientist, engineer, and professor, became the author of the first great textbook on mechanics of engineering. The first edition of his "Résumé des leçons données a l'École des Ponts et Chaussées sur l'application de la Mécanique a l'établissement des constructions et des machines", was published in 1826, and the second, revised edition in 1833. The book appeared also in an Italian translation. It is evidence of the great influence of this book, and of the high esteem in which it was held, that Barré de Saint-Venant, one of the great classics in the theory of elasticity, undertook, a number of years after the death of Navier, the preparation of a third, annotated edition, which appeared in final form in 1864.*

Navier, in the preface to the first edition, expressed well the purpose of structural analysis by comparing the stagnancy resulting from design by imitation with the progress which scientific methods make possible. The book, eminently practical, draws extensively on the experimental material available at the time. Of importance is the treatment, new because of its completeness, of the subject of beams. Stresses are determined for any cross-section, and the deflections are found by the method of double integration. This treatment is based on Bernouilli's assumption, often named "Navier's hypothesis," that the plane cross-section remains plane after bending. It was an accomplishment of investigators in the theory of elasticity of a later day, especially Saint-Venant and Pearson, to show that the errors introduced by this approximate assumption are only small for beams of ordinary dimensions. The result is that Navier's treatment of beams has become justly traditional. Although some additions have been made to it, this treatment holds its place in structural analysis in engineering of the present day.

Navier dealt competently in his "Mécanique" with subjects such as earth pressures, stability of masonry arches, and timber structures.

He treated particular cases of continuous beams, and he gave an analysis of the deflections of curved beams with a not too sharp curvature. He applied the latter analysis to some cases of two-hinged elastic arches. In 1825, Navier had published a note on the statics of bodies supported at more than three points.† He made therein the observation that a statically indeterminate problem becomes statically determinate if one takes into account conditions of elasticity. Except for some cases of statically indeterminate beams, which had been analyzed before, this note, in connection with the several cases treated in the book, represents, as far as is known, the beginning of the analysis of elastic, statically indeterminate structures. The same problems can be solved more easily by modern methods, but Navier's solutions are legitimate.

Navier applied the assumption of the plane cross-section remaining plane to torsion. This is proper in the case of a circular section, but improper in the case of non-circular sections. It was Saint-Venant who accomplished the difficult task of producing an exact theory of torsion for non-circular sections, and he inserted this theory in the third edition of the "Mécanique".

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* This third edition, besides the additions to the text, contains an account of the life of Navier and a general historical introduction.

† Navier, "Sur des questions de statique dans lesquelles on considère un corps supporté par un nombre de points d'appui surpassant trois," *Bulletin des Sciences* à la Société Philomatique, 1825, p. 35.

Navier's book is written with admirable clarity. It is no wonder that it exerted a great influence.

Two more of the works of Navier should be mentioned. In 1820, he submitted to the Paris Academy a paper on the flexure of slabs. Lagrange had obtained in 1811 a differential equation for this type of flexure. Navier gave an accurate solution for the case of a rectangular slab simply supported on four sides. The other work to be mentioned is a report of 1823 on his examination of English suspension bridges. It is significant that he tried to solve problems of the vibrations of the bridge.

The French Tradition

Lamé (1795-1870) and Clapeyron (1799-1864) were French engineers, who were for a time in Russian service. They contributed some papers jointly, especially one on the theory of internal equilibrium of solids, which was published by the Paris Academy.* This paper contains a number of definite applications of the theory of elasticity. Lamé published, in 1852, a volume of lectures on the theory of elasticity.† His formulas for the stresses in thick cylinders with internal pressure are well known. Clapeyron established an important formula, named after him, for the internal energy of an elastic body, due to the stresses.

In a short paper published in 1857‡ Clapeyron presented the famous "theorem of three moments" by which continuous beams may be analyzed conveniently. The paper has the appearance of a preliminary announcement of a longer paper, which, however, was never published. The form is rather interesting. The author begins by referring to the immense capital invested in railroads and the resulting growth of the problems of the engineers. He mentions a particular bridge with continuous girders, and says that practice here, as so often, has preceded theory. He states the formula, and shows by an example how to apply it.§

The great classic in the theory of elasticity, Barré de Saint-Venant (1797-1886), has been mentioned already in connection with the third edition of Navier's "Mécanique." Saint-Venant was an engineer and a mathematician. Most famous is his work on the subject of torsion.‖ Saint-Venant gave exact solutions for a number of definite cross-sections. Knowing the needs of engineers, he did not satisfy himself with establishing procedures of computation, or solutions in terms of infinite series, but he computed numerical coefficients, and represented many of the results graphically in a manner readily intelligible to any trained engineer.

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* Mémoires présentés par divers Savants, 1833.

† Lamé, "Leçons sur la théorie mathématique de l'élasticité des corps solides," Paris, 1852, Second Edition, 1866.

‡ *Comptes Rendus;* Paris Academy, v. 45, p. 1076.

§ The theorem of three moments is frequently named after Clapeyron, through whom it became generally known. It was given earlier, however, by Bertot, in *Comptes Rendus* de la Société des Ingénieurs Civils, 1855, p. 278. A more general form, applicable not only to a uniform load on each span, but to any type of load, was given later by Bresse.

‖ "Mémoire sur la torsion des prismes, etc," pub. by the Paris Academy in "Mémoires des Savants Etrangers," 1855, pp. 233–560.

One may explain the existence of a French tradition in structural analysis by referring to Navier and Saint-Venant. This tradition was upheld by disciples of Saint-Venant, among whom should be mentioned Boussinesq, H. Resal, Lévy, and Flamant. The researches of Boussinesq in a variety of fields of the theory of elasticity, theoretical and applied, as well as in the theory of earth pressures, are of considerable depth. H. Resal originated in 1862 an acceptable approximate theory of sharply curved beams (or hooks)*. Lévy wrote an extensive and valuable treatise, "Statique graphique" (Paris, 1874, Second Edition, 4 vol., 1886-88). And Flamant wrote treatises on structural statics and strength of materials.†

It was in France in 1894 that E. Coignet and N. de Tédesco, in a report to the Société des Ingénieurs Civils, originated the method which is now commonly used for computing nominal stresses in reinforced concrete beams for the purpose of design. This computation is based on neglect of horizontal tensile stresses in the concrete, on a linear relation otherwise of stresses and strains, and on the assumption that the plane cross-section remains plane.

A. Mesnager is a prominent contemporary representative of the French tradition. He introduced and made practicable the analysis of a complex structure by applying polarized light to a transparent model.‡ He has made important contributions to the theory of flexure of slabs.§

ENGLAND

France had Navier; Great Britain produced W. J. M. Rankine (1820-72). As Professor of Civil Engineering and Mechanics in the University of Glasgow, Rankine wrote remarkable textbooks in various fields of engineering. The first edition of his "Manual of Applied Mechanics" was published in London in 1858, the nineteenth edition in 1914. This book stands side by side with Navier's "Mécanique" as a great textbook on mechanics of engineering. New in this book is the determination of the distribution of shearing stresses in a beam by the analysis now current.‖ Another mark of progress is the introduction of the formula, which has been named after Rankine, for the strength of columns. Although not perfect, this formula has been of great service. Rankine credits the formula to Gordon. It had been proposed previously by F. Schwarz in 1854.

Rankine treated the problem of earth pressures from the point of view of a plane state of stress. His solution compares in importance with that of Coulomb. The two theories agree in important cases.

Rankine wrote numerous scientific papers, some of which deal with the theory of elasticity.

To understand the English tradition one must refer to Rankine; to his predecessors, Hodgkinson on the practical, experimental side, and the great mathematicians, Green and Stokes, on the side of the mathematical theory of elasticity; also to two great physicists of the Nineteenth Century, Maxwell and Lord Kelvin; and to Todhunter and Pearson, authors of the monumental "History of the Theory of Elasticity" (Cambridge, 1886-93). The practical problems of structural mechanics have been represented during recent years, for example, by A. Morley, author of noteworthy treatises on "Strength of Materials" and "Theory of Structures"; and the mathematical theory of elasticity has an eminent contemporary representative in A. E. H. Love, author of a great treatise on this subject (first edition, 1892-93; third edition, 1920).

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* *Annales des Mines,* 1862, p. 617.

† A. Flamant, "Stabilité des constructions; Résistance des matériaux," Paris, 1886; Second Edition, 1897.

‡ *Annales des Ponts et Chaussées,* 1913, p. 133.

§ A series of papers in *Comptes Rendus,* Paris Academy, 1916–19.

‖ First Edition, 1858, p. 338.

Early Works on Trusses

The prominent American engineer, the late Squire Whipple, Hon. M. Am. Soc. C. E., inventor of the Whipple truss, published a work on "Bridge Building"* in 1847. This work contains an analysis, which, so far as is known, is the first legitimate analysis of the jointed truss. His style reveals a clear mind. He accomplished his task by means of the simplest possible principle of statics, that of the force resolved into a vertical and horizontal component. The American engineer, Herman Haupt, evidently independent of Whipple, followed in 1851 with an attempt to analyze trusses.† One finds in his work rather a mixture of sound and unsound thinking.

In 1863, August Ritter, in Germany, published his famous method of sections by which each stress is found by expressing the moments of all forces on one side of the section with respect to a suitable center.‡ This method holds its place at the present day as a most effective way of determining stresses in trusses.

Graphical Methods

Graphical methods have played an important part in structural analysis. A "graphical statics" exists.

Particular graphical methods were used fairly early. Poncelet, for example, proposed, in 1840, his elegant method (frequently credited to G. Rebhann in 1871) for determining the earth pressures which exist according to Coulomb's theory. The English clergyman and Professor of Natural Philosophy, H. Moseley, a disciple of Poncelet, introduced in 1833 the line of pressure of the masonry arch.§

C. Culmann, born in 1821, was a Professor at the Polytechnikum in Zürich, Switzerland, from 1854 until his death in 1881. While Varignon in his "Nouvelle mécanique" of 1725 had studied the polygon of equilibrium of a string, it is reasonable to state that graphical statics as an independent subject came into existence when Culmann, by emancipating himself from the idea of a material string, made the string polygon a tool of analysis. Culmann found many uses for the string polygon; for example, in the analysis of masonry arches; and in determining the moment of inertia of an area. In the analysis of trusses he introduced a method of sections similar to that of Ritter, except that the three stresses in the section are found graphically as components of the resultant of the external forces on one side of the section. Culmann's remarkable book on structural mechanics, published in 1864-66, has the title, "Graphische Statik".

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* See S. Whipple, "Elementary and Practical Treatise on Bridge Building," an enlarged and improved edition of the author's original work; Second Edition, N. Y., 1873. Reference is made in the preface to the "Original Essays" of 1847. See, also, H. G. Tyrrell, "History of Bridge Engineering" (published by the author, Chic., 1911), p. 166; or the historical section in a very excellent book by J. I. Parcel and G. A. Maney, "An Elementary Treatise on Statically Indeterminate Stresses," N. Y., John Wiley & Sons, 1926, p. 352.

† See Herman Haupt, "General Theory of Bridge Construction," N. Y., 1869. Pt. I of this book represents Haupt's early work.

‡ A. Ritter, "Elementare Theorie und Berechnung eisener Dach- und Brückenkonstruktionen," Hannover, 1863.

§ H. Moseley, "On the Equilibrium of the Arch" (paper read 1833), *Transactions*, Cambridge Philosophical Soc., v. 5, 1835. p. 293; or "The Mechanical Principles of Engineering and Architecture," Lond., 1843, p. 403.

J. Clerk Maxwell (1831-79), the great English physicist, invented, in 1864,[*] the stress diagram for trusses which unites into a single figure all the individual force polygons. Cremona developed the method further. The diagram is often named after Maxwell and Cremona. It found its place quickly in graphical statics of engineering.

Graphical statics owes a great deal to Otto Mohr (1835-1918), for a number of years Professor at the Technische Hochschule in Dresden, Germany. In 1868 he published his method of finding deflections of a beam by use of string polygons. He established on the same occasion the closely allied method of computing deflections as if they were bending moments.[†] The late C. E. Greene, M. Am. Soc. C. E., at the University of Michigan, about 1873, discovered a related principle of moments of areas by which one may find the deflection of any point of a beam, measured from a tangent to the elastic curve. H. Müller-Breslau, in 1885, gave Mohr's principle a general formulation,[‡] with the application extended to trusses; the general principle includes Greene's principle as a special case.

Mohr invented a use of string polygons in determining stresses due to bending combined with compression when the material has no tensile resistance. C. Guidi, in Italy, showed in 1906 that this method, with a small modification, can be applied to a reinforced concrete member with any symmetrical cross-section.

Mohr devised an elegant graphical representation of the three-dimensional state of stress at a point, and he invented the circle of inertia for determining moments of inertia of an area with respect to different axes through the same point. Land showed later[§] that the latter diagram may be applied advantageously in analyzing a plane state of stress at a point.

E. Winkler introduced the influence line in 1867.

Williot proposed, in 1877, his diagram by which the displacements of all points of a truss are obtained as vectors.[||]

After Culmann's death, W. Ritter continued his work at the Polytechnikum in Zürich. Ritter's treatise[¶] possesses distinction and originality. The ellipse of elasticity, proposed and used extensively by W. Ritter, has interested many. No one will deny its value in defining qualitatively the nature of the stiffness of a flexible connection between two elements of a structure.

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[*] *Philosophical Magazine,* (4), v. 27, 1864, p. 250.

[†] See the collection of papers by Mohr, re-edited by himself, "Abhandlungen aus dem Gebiete der technischen Mechanik," Berlin, 1906, Second Edition, 1914.

[‡] H. Müller-Breslau, "Beitrag zur Theorie des Fachwerks," *Zeitschrift* des Architekten- und Ingenieur-Vereines zu Hannover, v. 31, 1885, p. 418.

[§] *Zeitschrift* des Vereines deutscher Ingenieure, 1895, p. 1551.

[||] Williot, "Notations pratiques sur la statique graphique," Publications scientifiques industrielles, 1877.

[¶] "Anwendungen der graphischen Statik nach Professor Dr. C. Culmann," 4 vol., Zürich, 1888–1906.

Some graphical methods refer to continuous beams. Two should be mentioned, one originated by Mohr, the other by Claxton Fidler. Mohr's method is based on properties of the tangents to the elastic curve. Culmann, Winkler, and especially W. Ritter[*] took up the idea and extended its use. The other method, which has turned out to be simpler, may be interpreted as a graphical solution of the equation of three moments. Claxton Fidler originated it in 1883, Müller-Breslau gave it in an improved form in his "Graphische Statik der Baukonstruktionen", and Ostenfeld developed it further.[†] L. H. Nishkian and D. B. Steinman, Members, Am. Soc. C. E., have presented the same method.[‡]

Statically Indeterminate Structures

The theories of statically indeterminate structures may be grouped according to certain leading principles, essentially four.

Maxwell, the great English physicist, originated the first of these in 1864,[§] the same year that he invented the stress diagram for trusses. It was on this occasion that he gave the famous theorem of reciprocal deflections which is named after him. In dealing with the statically indeterminate truss he expressed the condition of geometrical coherence by a set of equations in which the redundant stresses are the variables. Mohr, in 1874, derived the same equations by a direct use of the principle of virtual work. The equations are named frequently after Maxwell and Mohr. Mohr's papers on the subject[||] established firmly the applicability of the principle. He included the effects of variations of temperature. Müller-Breslau, in his weighty treatises,[¶] included the effects of settlements of the supports and of incorrect initial dimensions of the members, and he extended the method so as to make it apply to solid frames as well as to trusses.

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[*] See v. 3 (published 1900) of his "Graphische Statik," previously mentioned. This volume has the sub-title, "Der kontinuierliche Balken."

[†] T. Claxton Fidler, *Minutes of Proceedings*, Inst. C. E., v. 74, 1883, p. 196. See, also, his "Practical Treatise on Bridge Construction," Lond., 1887, Fifth Edition. 1924, Chapter 9. H. Müller-Breslau gave the method in a more general form in an article, "Ueber einige Aufgaben der Statik, welche auf Gleichungen der Clapeyronschen Art führen," *Zeitschrift für Bauwesen*, v. 41, 1891, Columns 103–128, and thereafter in his "Graphische Statik der Baukonstruktionen," v. 2, sub-vol. 1, Leipzig, 1892, p. 357 (Fifth Edition, 1922, p. 406); also, v. 2, sub-vol. 2, 1908, p. 37 (Second Edition, 1925, p. 93). A Ostenfeld's treatment of the subject may be found in his "Teknisk Elasticitetslære," Second Edition, Copenhagen, 1905, p. 176; Fourth Edition, 1924, p. 230; in his "Teknisk Statik," v. 2, Copenhagen, 1903, p. 92; Third Edition, 1925, p. 87; and in two papers, "Graphische Behandlung der kontinuierlichen Träger mit festen, elastisch senkbaren oder drehbaren und elastisch senk- und drehbaren Stützen." *Zeitschrift* für Architektur und Ingenieurwesen, v. 51, 1905, Columns 47–66, and "Graphische Behandlung der kontinuierlichen Träger mit elastisch senkbaren Stützen," *loc. cit.*, v. 54, 1908, Columns 57–78.

[‡] *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 1.

[§] J. Clerk Maxwell, "On the Calculation of the Equilibrium and Stiffness of Frames," *Philosophical Magazine*, (4), v. 27, 1864, pp. 294–299.

[||] See the collection of papers by Otto Mohr," "Abhandlungen aus dem Gebiete der technischen Mechanik," Second Edition, Berlin, 1914, pp. 390–479, especially his bibliographical notes at the end of the paper.

[¶] "Die neueren Methoden der Festigkeitslehre und der Statik der Baukonstruktionen," Leipzig, 1886 (Fifth Edition, 1924), and "Die graphische Statik der Baukonstruktionen," v. 2, sub-vol. 1, Leipzig, 1892 (Fifth Edition, 1922).

H. Müller-Breslau (1851-1925), for many years a Professor at the Technische Hochschule, in Berlin, is indeed one of the most distinguished figures in the history of structural statics.[*] In addition to the many other contributions, he is responsible for the systematizing of what may be called the second leading principle in the analysis of statically indeterminate structures.[†] The procedure may be looked upon as a variation of that of Maxwell. The essential variables, again, are the redundant stresses (or measures of combined states of stresses, such as bending moments). The linear equations expressing the condition of geometrical coherence are obtained by superposition of displacements due to the individual loads. The coefficients in these equations are displacements due to unit loads. By expressing the individual terms by the principle of virtual work, the equations assume the form obtained by Maxwell and Mohr. The general form, however, in which Müller-Breslau stated these equations leaves the way open to obtain the terms by any method which is available for determining displacements. Müller-Breslau pointed out the usefulness, in this connection, of Maxwell's theorem of reciprocal deflections, and thus he came to the important conclusion that the influence line for a redundant stress or bending moment may be interpreted as a diagram of deflections. During recent years George E. Beggs, M. Am. Soc. C. E., at Princeton University, using this property of the influence line, has developed an experimental method which appears promising in connection with frames which are many times statically indeterminate; he obtains influence lines by observing deflections of models made of paper or celluloid.[‡]

The third of the leading principles is that of least work. This principle was established brilliantly during the Seventies by the Italian engineer A. Castigliano (1847-84). It should be mentioned that the Italian Count who later became a General, L. F. Menabrea, had stated the principle in 1858.[§] The method, however, is named fairly after Castigliano rather than after Menabrea, whose proof was erratic. Castigliano presented his method in a remarkable book, "Théorie de l'équilibre des systèmes élastiques et ses applications" (Torino, 1879). The unknown quantities determined first are the same as those entering in the equations due to Maxwell and Mohr. Müller-Breslau contributed greatly toward making Castigliano's work known; and E. S. Andrews, in England, performed a service by translating Castigliano's book into English, under the title "Elastic Stresses in Structures" (London, 1919).

The fourth and last of the leading principles has been used on a number of occasions, but was given a general formulation only recently by A. Ostenfeld, in Denmark. By this principle the unknown quantities upon which the analysis depends primarily, are deformations. The equations of elasticity are of the same form as Maxwell's and Mohr's equations, except that loads and deformations have changed places. Ostenfeld published a book on this subject in 1925, entitled "Die Deformationsmethode". Such methods as the slope-deflection method, by which slopes and transverse deflections are the essential unknowns, may be interpreted as examples of application of this general principle.

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[*] A list of Müller-Breslau's publications may be found in *Zeitschrift für angewandte Mathematik und Mechanik*, v. 5, 1925, p. 277.

[†] "Die neueren Methoden der Festigkeitslehre und der Statik der Baukonstruktionen," 1886.

[‡] *Proceedings*, Am. Concrete Inst., v. 18, 1922, p. 58.

[§] *Comptes Rendus*, Paris Academy, v. 46, 1858, p. 1056.

Among the great textbooks dealing with statically indeterminate structures, with the greatness measured by quality and influence, should be mentioned the following: Müller-Breslau's "Graphische Statik der Baukonstruktionen"; Ostenfeld's "Teknisk Statik" (the last edition of which includes the deformation method, see v. 2, Third Edition, Copenhagen, 1925); and the book by the late J. B. Johnson, the late C. W. Bryan, and F. E. Turneaure, Members, Am. Soc. C. E., entitled "Modern Framed Structures" (Ninth Edition, v. 2, New York, 1911).

Germany, with leaders like Mohr and Müller-Breslau, played an important part in the development of this subject.

THEORY OF ELASTICITY

The theory of elasticity has been applied to practical problems in many ways during recent years. A. Föppl (1854-1924), not least through his wonderful book, "Technische Mechanik", the fifth volume of which deals with the theory of elasticity (v. 5, First Edition, Leipzig, 1907), doubtless exerted a great deal of influence in this respect.* In 1920, he and his son, L. Föppl, who later became his successor as Professor at the Technische Hochschule, in Munich, published jointly a two-volume theory of elasticity for engineers, with the peculiar title "Drang und Zwang".

W. Ritz in a notable paper published in 1908† gave emphasis to a principle of minimum which lends itself particularly well to approximate solutions. Admissibility of approximate methods extends the field of accessible problems greatly. Ritz's principle of minimum is based on variation of shape, whereas Castigliano's principle is based on variation of the state of stress. Approximate investigations based on the minimum of energy by variation of shape are described frequently as analysis by Ritz's method.

A. and L. Föppl's "Drang und Zwang" contains many examples of this type of analysis.

Important work has been carried out during recent years on torsion, especially by L. Prandtl and A. Föppl.‡ Prandtl discovered in 1903 the "soap-film analogy", by which problems of torsion may be solved. He showed that the equations governing the state of stress in torsion are the same as those governing certain properties of a soap film deflected by a pressure from one side. Thus, one may transfer solutions by analogy from one field of mechanics to another. The soap film, being easy to visualize and to produce experimentally, and being subject to fairly simple laws of mechanics, has proved to be very useful in this respect.

Much work has been done during the last ten or fifteen years on the subject of the flexure of slabs. Two names should be mentioned: A. Nádai in

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* A list of A. Föppl's technical and scientific publications, including eighty-five items, may be found in *Zeitschrift für angewandte Mathematik und Mechanik*, v. 4, 1924, p. 530.

† *Crelles Journal*, v. 135, 1908.

‡ See A. and L. Föppl, "Drang und Zwang," v. 2, 1920.

Germany and B. Galerkin in Russia. Nádai wrote an excellent book, "Die elastischen Platten" (Berlin, 1925), in which many of the recent investigations are described.

H. Reissner, in Berlin, and E. Meissner, in Zürich, have made important contributions to the theory of domes and shells.*

RECENT DEVELOPMENTS

The plastic stage, occurring when certain stresses exceed the proportional limit or the yield point, has been the object of some theoretical studies during the more recent years.

The action of a column of ductile material presents a problem of this kind. If one wishes to understand this action, it is necessary to consider the changed conditions of stiffness beyond the proportional limit. Engesser and Considère discussed the mechanics of this case some time ago. A fully satisfactory treatment of this subject, accompanied by careful tests, came in 1910 from the hand of Th. v. Kármán, then in Göttingen.† His contribution is one of the most important in the whole vast literature on the subject of columns.

Prandtl and, after him, Hencky have succeeded during the last few years in analyzing the state of stresses and deformations occurring in certain cases of plastic action when one body penetrates into another.‡ Nádai has analyzed cases of torsion beyond the proportional limit.§

Charles Terzaghi, M. Am. Soc. C. E., has attacked the fundamental problems of the mechanics of soils.‖ Strikingly interesting is the mutual influence found here of the state of stress and the movement of water. A problem in strength of materials becomes hydraulics.

Finally, may be mentioned an exceptional piece of work, theoretical and experimental, by A. Brandtzæg, presented in a thesis at the University of Illinois, in 1926.¶. The subject is compressive strength of concrete. Following and developing a procedure introduced by R. Böker in 1913,** he applied methods of statistical mechanics to a solid consisting of crystalline parts with planes of weakness distributed by laws of chance in all directions. He succeeded in this way in accounting for a part of the curved portion in the stress-strain diagram, and he explained a number of the phenomena which he found experimentally in cases of three-dimensional states of stress. Here, surely, the subject of strength of materials is in a new phase of development.

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* Concerning this and other modern applications of the theory of elasticity, see the article by L. Föppl, "Neuere Fortschritte der technischen Elastizitätstheorie," *Zeitschrift für angewandte Mathematik und Mechanik*, v. 1, 1921, pp. 466–481.

† *Mitteilungen über Forschungsarbeiten aus dem Gebiete des Ingenieurwesens*, Heft 81, 1910.

‡ L. Prandtl, *Zeitschrift für angewandte Mathematik und Mechanik*, v. 1, 1921, p. 15; H. Hencky, *loc. cit.*, v. 3, 1923, p. 241; L. Prandtl, *loc. cit.*, p. 401, and *Proceedings*, First International Congress for Applied Mechanics, Delft, 1924 (pub. 1925), p. 43.

§ *Zeitschrift für angewandte Mathematik und Mechanik*, v. 3, 1923, p. 442.

‖ Series of articles in *Engineering News-Record*, November-December, 1925; also, *Proceedings*, First International Congress for Applied Mechanics, Delft, 1924 (pub. 1925), p. 288.

¶ The theoretical part of this investigation has now been published under the title, "Failure of a Material Composed of Non-Isotropic Elements; an Analytical Study with Special Application to Concrete", Det kgl. Norske Videnskabers Selskabs Skrifter, 1927, Nr. 2, Trondhjem, Norway, 1927.

** R. Böker, "Die Mechanik der bleibenden Formänderung in kristallinisch aufgebauten Körpern," *Forschungsarbeiten auf dem Gebiete des Ingenieurwesens*, Heft 175–176, Berlin, 1915; dissertation with the same title, Berlin 1913.

The Present

It is by no means discouraging to consider the state of affairs in structural mechanics of the present day. Group enterprises of many kinds are promoting research and distributing knowledge. Think, for comparison, of the situation 150 years ago when only a few learned academies performed these functions. Now engineering societies, schools of engineering, and some periodicals do much to make research possible. One periodical of this kind should be mentioned especially. It is the *Zeitschrift für angewandte Mathematik und Mechanik,* published since 1921. Its editor is a man of unusual caliber, Professor R. von Mises, of Berlin. The German periodical, *Der Bauingenieur,* published since 1920, also deserves notice.

In the spring of 1924, largely due to the initiative of Professor C. B. Biezeno, of Delft, a "First International Congress for Applied Mechanics" was held in Delft, Holland. The *Proceedings* published the following year contain much material of practical and scientific value. A Second Congress was held in September, 1926, in Zürich, Switzerland.

A recent enterprise of great value is the "Encyklopädie der mathematischen Wissenschaften", of which Volume IV, which consists of four sub-volumes, has the title "Mechanik". Volume IV has been completed except for a part of Sub-Volume 2. The plan of this encyclopedia is to summarize all the important work of the field.* Another similar enterprise has been started very recently under the title "Handbuch der physikalischen und technischen Mechanik".†

Among the individuals representing structural analysis at the present day, there are some stars of considerable magnitude. Among them four may be mentioned: L. Prandtl and Th. v. Kármán, in Germany; A. Ostenfeld, in Denmark, and S. Timoshenko, formerly in Russia, but since 1922 in the United States. They are men of broad scholarship and creative imagination.

The work of Dr. Timoshenko brings to attention the Russian tradition, of which he is a contemporary representative. One may trace this Russian tradition back to the days of Euler and his influence through the St. Petersburg Academy. Dr. Timoshenko's presence in this country is a reminder, at the same time, of the international character of the science of structural mechanics.

Structural analysis is, of course, intimately allied with and dependent on structural testing. The United States has excelled in the latter field. The grand scale of this experimental work, and the determination to find out, have aroused the admiration of Europeans. In structural engineering this country has also excelled. In structural analysis, it has done well, yet the great ideas have come mainly from Europe.

Hope for future advance in structural analysis lies in the fact that the subject, international as it is, fascinates many in all parts of the world.

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* The article by L. Henneberg, in Sub-Vol. 1, and those by Th. v. Kármán, M. Grüning, and K. Wieghardt, in Sub-Vol. 4, are of especial interest in connection with structural analysis.

† Edited by F. Auerbach and W. Hort and published in Leipzig, Germany.

DISCUSSION

S. TIMOSHENKO,* ESQ. (by letter).—It is a consequence of the ancient structural project at Babel (as the story goes), that linguistic barriers have hampered the progress of science. These barriers have become of slight concern to the English speaking peoples with reference to many of the languages, especially French and German, but, unfortunately, the Russian barrier is not so easily surmounted. The writer takes this opportunity to help remedy this, and to elaborate the reference made to the Russian tradition.

The development of structural analysis in Russia begins with the work of Daniel Bernouilli, who was invited there in 1725 by Catherine I, and was appointed to the Professorship of Mathematics in the Academy of St. Petersburg. Bernouilli was the first to obtain the differential equation of the deflection curve of a prismatic bar. He arrived at this by a consideration of the transverse vibration of the bar.† To his inspiration may also be credited the investigation of elastic problems by L. Euler (1701-83) who in 1733 succeeded Bernouilli in the Chair of Mathematics at St. Petersburg. Euler's most important contribution to the theory of structure is his discussion of the buckling of columns subjected to axial compressive forces. He treated not only the case in which the load is applied at the top of the column, but also the effect of the weight of the column. He was also the first to investigate the bending of bars originally curved. He introduced the assumption that the bending moment in this case is to be taken proportional to the difference between the original and the new curvature. This assumption was proved later by considering the nature of the elastic forces.

At the beginning of the Nineteenth Century the development of the theory of structures in Russia was greatly influenced by two Frenchmen, Lamé and Clapeyron, who were professors in one of the oldest Russian engineering schools, "The Institute of the Engineers of Ways of Communications". This school was opened in 1807 and later had a prominent rôle in the development of engineering education in Russia. Professor Jouravski of this school was in charge of the building of the first railway bridges in Russia and was one of the first to develop methods of determining the forces in members of a truss. The Ministry of Ways of Communication was skeptical about his conclusion that in the Howe truss the forces in the verticals (bolts) produced by uniformly distributed load increase from the middle of the span to the supports. In order to demonstrate his theory, Jouravski used an interesting model. The verticals were replaced by strings and from the tones of these when plucked, the stress differences were readily shown.

He gave also the elementary theory of shearing stresses in beams, which is now found in all books on strength of materials.‡

As concerns curved bars, H. Golovin was the first to give an exact theory of the bending of curved bars of a rectangular cross-section.§ His work, published in Russian, remained unknown in Western Europe, and his solution was later rediscovered by L. Prandtl in Germany[*] and by Ribière in France.[†]

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* Ann Arbor, Mich.

† Commentarii, St. Petersburg Academy, 1741–43.

‡ *Annales des Ponts et Chaussées*, 1856, p. 328.

§ *Bulletin,* Technological Inst., St. Petersburg, 1881.

The theory of elastic stability was advanced considerably by the work of Professor F. Jassinsky of the previously mentioned Russian engineering school. He developed[‡] the theory of the buckling of bars having elastic lateral supports and applied this theory to the investigation of the stability of compressed chords of through span bridges. To Jassinsky engineers are indebted also for the correct solution of the problem of the buckling of bars compressed beyond the yield point. His books on the mathematical theory of elasticity and on the theory of structures also had a great influence on the development of the teaching of these sciences in Russian engineering schools.

Among the Russian engineers of the beginning of the Twentieth Century may be mentioned A. N. Kriloff, who made considerable progress in the theory of vibration of bridges and ships[§] and I. G. Boobnoff who became prominent because of his work in the theory of the structure of ships.[‖] He gave a very complete treatment of the bending of rectangular plates submitted simultaneously to hydrostatic pressure and to tension in the middle surface. Boobnoff was also the first to give an accurate solution of the problem of bending in rectangular slabs with built-in edges.

Alfred D. Flinn,[¶] M. Am. Soc. C. E. (by letter).—Professor Westergaard's condensed and well co-ordinated summary of the successive steps along the several important lines of structural analysis, should be stimulative and helpful to further progress. It is a convenient and interesting historic narrative and annotated bibliography; but it is more. Professor Westergaard has produced an orderly series of skillful, critical, and lucid analyses of the contributions of the several distinguished workers in the field, who have made modern methods of structural analysis feasible. This summary illuminates the subject, makes relationships apparent, and indicates need for further researches.

The paper also emphasizes the facts that modern engineering is truly a learned profession and that it demands a rigorous educational preparation. That much vaunted "horse sense", if untutored, is no longer safe or sufficient equipment for the design of present-day structures to which human lives, limbs, and fortunes are committed. Adequate preparation for the practice of such engineering demands, also, better teaching than has been offered by some engineering colleges in years gone by.

Jacob Feld,[**] Assoc. M. Am. Soc. C. E. (by letter).—In summarizing, in such compact form, the original contributions to the science of structural analysis, the author has presented a study which ranks in value far above the usual historical scientific paper. Although no attempt was made to explain the methods of attack of the men whose contributions are mentioned, the results as applicable to structural analysis as well as the effect of one man's work upon succeeding generations are clearly outlined. In spite of the enormous amount of historical data, the paper is written in a style which makes interesting reading. The addition of footnote references to the more important contributions makes this paper quite valuable as an outline for the research student in this field. It is to be hoped that it will be given widespread distribution among students and research workers.

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[*] See A. Föppl, "Technische Mechanik," Vol. 5, 1907, p. 72.

[†] *Comptes Rendus*, T. 108, 1889.

[‡] "Theory of Buckling," St. Petersburg, 1892.

[§] *Mathematische Annalen*, Vol. 61 (1905).

[‖] "The Strength of Ships," Vols. I and II, St. Petersburg, 1913.

[¶] Secy., United Eng. Soc., and Director, Eng. Foundation, New York, N. Y.

[**] Cons. Engr., New York, N. Y.

The paper deals entirely with theoretical analysis as applied to structural design. It would be interesting to have a parallel discussion of the experimental work contributed by a great number of the men whose names are mentioned by the author, as well as by others, which has been used to check the theories proposed and to put the study on a scientific basis.

The discussion deals chiefly with statics, that portion of structural analysis which is used mostly by the membership of the Society. However, a great number of the contributions mentioned are really only special cases of the general theoretical discussions of dynamic principles. Considerable simplification has resulted in the very complicated equations for the general solutions of most dynamic problems by the use of vector notation and vector processes. This is merely a method of reducing the mechanical labor in mathematical derivation or a new method of placing certain ideas in symbolic form. The resulting vector mechanics has been applied in the mechanical and electrical fields as well as in the aerodynamic field. However, the recent expansion of structural design to the work required in airplane and airship production practically necessitates the inclusion of the general dynamic principles in the list of contributions to the field of structural analysis. The application of stress analysis for non-homogeneous bodies, such as reinforced concrete, has found a fertile field in the design of airships of various combinations of rigid keels and non-rigid envelopes. It is found that a great number of very theoretical contributions in stress analysis are quite applicable to the actual design of such structures.

Although separated in the text, the work in elastic stress by Cauchy and by Lamé are complementary, and together they give the general solution of the magnitude (by Cauchy) and of the direction (by Lamé) of stress. The results, analytically, are quite complicated; considerable simplification has been introduced by Föppl by the use of vector notation.

The author has omitted a fairly important contribution to the subject of elasticity based on the so-called energy function. On this basis, Kirchhoff and Helmholz independently showed that an isotropic elastic body can be defined completely by two elastic constants, the modulus of shear or rigidity, and the bulk modulus of elasticity. Experimentally, the latter constant was studied by Karman.* His work on small cylinders subjected simultaneously to hoop and longitudinal pressures supplements his work on columns of ductile materials mentioned by the author. Following the same idea of the energy function, S. E. Slocum, M. Am. Soc. C. E., has derived general formulas for the shearing deflection of beams of arbitrary, variable, or constant cross-section and also a general formula for the torsional deflection.* Most of the relations in indeterminate structures can easily be deduced from the idea of the energy function.

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* *Zeitschrift des Vereines deutscher Ingenieure*, Vol. LV, 1911, pp. 1749-1757.

It is quite surprising to find how much repetition there is in names of contributors in the field of structural analysis and in that of soil mechanics.† Both subjects seem to start with the same man, Coulomb, although previous work by a great number of men had paved the way. Just as Navier, Cauchy, and Poisson founded the theory of elasticity, Navier in his generalization of Coulomb's theory in 1826 to the cases of surcharged walls and also to the cases of soils containing cohesive as well as frictional resistances, and Cauchy and Poisson (whose works were the bases of the earth pressure theory by Maurice Levy, Saint-Venant, and Boussinesq), can be classed as the founders of the present two schools of soil study.

It is seldom realized how much progress in scientific study has been lost because of poor translation or the inclusion of the translator's own ideas into a book which bears the name of the original author. As a result, discrepancies will often be found in the reported contributions of certain men to any one subject, and it is quite important that the original references be consulted for a true evaluation. In this respect, Professor Westergaard's paper is especially good and, therefore, reflects considerable credit upon its author.

H. M. Westergaard,‡ M. Am. Soc. C. E. (by letter).—The writer has read with interest and appreciation the discussions by Professor Timoshenko, and Messrs. Flinn and Feld. Indebtedness is expressed to Professor Timoshenko for his account of the Russian tradition.

The writer does not understand the remark by Mr. Feld about Cauchy and Lamé. His reference to an investigation by S. E. Slocum, M. Am. Soc. C. E., published in 1911, to determine the influence of the shearing stresses in beams upon the deflections, by consideration of the energy, may be supplemented by the statement that W. Ritter solved the problem in 1888 by a simpler consideration of the energy, and that R. Land presented a particularly satisfactory simple solution of the whole problem in 1894.§

Professor Timoshenko referred to the work of Boobnoff in the theory of the structure of ships, and Mr. Feld mentioned the structural problems in aeronautical design. It is significant that the applications of structural mechanics extend to many fields of engineering. In aeronautical design the emphasis on economy of weight has led to new problems, involving, especially, the possibility of failure by buckling.‖

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* *Journal*, Franklin Inst., April, 1911, and July, 1912.

† "The History of the Development of Lateral Earth Pressure," by Jacob Feld, *Proceedings*, Brooklyn Engrs. Club, January, 1928, pp. 61-104.

‡ Prof. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

§ W. Ritter, "Anwendungen der graphischen Statik," v. 1, 1888, p. 148; R. Land, "Einfluss der Schubkräfte auf die Biegung statisch bestimmter und die Berechnung statisch unbestimmter gerader vollwandiger Träger," *Zeitschrift für Bauwesen*, v. 44, 1894, Columns 611–626.

‖ For example, on the subject of wing-beams, see, H. Reissner and E. Schwerin, "Die Festigkeitsberechnung der Flugzeugholme," Jahrbuch der Wissenschaftlichen Gesellschaft für Luftfahrt, v. 4, 1916; H. Müller-Breslau, "Graphische Statik der Baukonstruktionen," v. 2, sub-vol. 2, Second Edition, 1925, pp. 647 and 661; and Joseph S. Newell, Jun. Am. Soc. C. E., "The Design of Airplane Wing-Beams," *Proceedings*, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 725.

Professor H. Reissner, of Berlin, Germany, in a letter to the writer, expressed the view that certain chapters in the history of structural analysis might well have been included in the paper, or dealt with more fully. He mentioned the following subjects: Vibrations,[*] buckling,[†] airplanes, and experimental devices for structural analysis. The writer agrees with Professor Reissner, but wishes to add that many more subjects might have been included.

Referring to the experimental devices, the introduction of the use of polarized light by A. Mesnager, for the purpose of determining stresses, was mentioned in the paper, but it should have been stated that E. G. Coker has done eminent, extensive work with this method[‡] The use of models with the devices invented by George E. Beggs, M. Am. Soc. C. E., was mentioned in the paper. The soap film analogy according to L. Prandtl, for the study of torsion of non-circular members, was also mentioned. This analogy has been supplemented by a roof-and-membrane analogy indicated by A. Nádai, whereby one may study, experimentally if one wishes, the progress of plastic action or yielding that must take place in a non-circular member subject to torsion.[§] Lydik S. Jacobsen has devised a simple and effective method of determining the stresses produced in a circular shaft with variable diameter by measuring electric potentials in a plate of non-uniform thickness.[‖]

The subject of secondary stresses in trusses might well have been discussed in the paper. At this place, it will be sufficient, however, to refer to the notable paper on the subject by Cecil Vivian von Abo, Assoc. M. Am. Soc. C. E., which contains an historical discussion.[¶]

An important principle, which was not mentioned in the paper, was stated by Melan in 1888, and applies to suspension bridges.[**] A heavy chain possesses flexural resistance by virtue of its weight. Applying this structural principle to the bridge, one finds that the stiffening trusses are aided by the dead load.

In concluding, the writer wishes to call attention to a recent development in the theory of beams. In order to deflect vertically, without twisting, a channel used as a beam with the web vertical should be loaded so that the resultants of the loads lie in a certain vertical plane. This plane does not contain the centers of gravity of the cross-sections, nor the centers of the web, but is found outside the beam behind the web. The strikingly simple explanation was found by A. Eggenschwyler* and R. Maillart.† In the cross-section one finds vertical shearing stresses in the web and horizontal shearing stresses in the flanges. The resultant of the horizontal shearing stresses is a couple, which, combined with the resultant of the vertical shearing stresses, gives as the resultant of all the shearing stresses a vertical force lying in a plane outside the beam. The resultants of the loads, obviously, must be contained in this plane, if the beam is to deflect without twisting. Tests‡ with steel channels have shown this conclusion to be accurate, but one may also perform the experiment convincingly with a model made of cardboard. It is strange that a structural principle so simple and important, and so easy to demonstrate, should have remained unnoticed until only a few years ago.

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[*] It is proper to mention in this connection the following paper by H. Reissner: "Schwingungsaufgaben aus der Theorie des Fachwerks," *Zeitschrift für Bauwesen*, v. 53, 1903, Columns 135–162. On the general subject of vibrations, see, W. Hort, "Technische Schwingungslehre," 1922 (bibliography on vibrations of trusses, p. 802), and S. Timoshenko, "Vibration Problems in Engineering," 1928.

[†] A bibliography on the subject of buckling may be found in *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 650. The following more recent investigation should be mentioned: R. v. Mises, "Ueber die Stabilitätsprobleme der Elastizitätstheorie," *Zeitschrift für angewandte Mathematik und Mechanik*, v. 3, 1923, p. 406. The same volume contains four other notable papers on buckling.

[‡] See, for example, E. G. Coker, "Some Engineering Problems of Stress Distribution," *Proceedings*, First International Congress for Applied Mechanics, Delft, 1924 (pub. 1925), p. 18.

[§] A. Nádai, "Der bildsame Zustand der Werkstoffe," 1927, p. 97.

[‖] Lydik S. Jacobsen, "Torsional-Stress Concentrations in Shafts of Circular Cross-Section and Variable Diameter," *Transactions*, Am. Soc. Mech. Engrs., v. 47 (1925), p. 619.

[¶] Cecil Vivian von Abo, "Secondary Stresses in Bridges," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1. See, also, Z. Bazant, "Influence du système de triangulation sur les efforts secondaires," Académie Masaryk du Travail, *Publication Scientifique No. 15a*, Prague, 1923, and "Les efforts secondaires des poutres en treillis," *Loc. cit., Publication Scientifique No. 87a*, Prague, 1926.

[**] Leon S. Moisseiff, "The Towers, Cables and Stiffening Trusses of the Bridge Over the Delaware River between Philadelphia and Camden," *Journal*, Franklin Inst., October, 1925.

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* *Schweizerische Bauzeitung*, v. 76, 1920, pp. 206 and 266; *Eisenbau*, 1921, p. 207; *Zentralblatt der Bauverwaltung*, v. 41, 1921, p. 501; *Der Bauingenieur*, v. 3, 1922, p. 11.

† *Schweizerische Bauzeitung*, v. 77, 1921, p. 195; v. 83, 1924, p. 109; see, also, A. Ostenfeld, "Teknisk Elasticitetslære," Fourth Edition, 1924, p. 404; A. and L. Föppl, "Drang und Zwang," v. 2, Second Edition, 1928, p. 122.

‡ Especially by Bach; see the papers by Eggenschwyler and Maillart referred to previously.

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EVALUATION OF WATER RIGHTS*

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With Discussion by Messrs. Joseph Jacobs, Fred H. Tibbetts, Charles R. Hedke, Fred C. Scobey, Lyman E. Bishop, R. L. Parshall, S. H. Stivers, C. E. Grunsky, and John E. Field.

Synopsis

The methods and conclusions given in this paper are based on twenty years' experience in making appraisals wherein the value of water rights was an important part. This has involved water values in excess of $10 000 000.

Appraisals on New Projects

Somewhat related to an appraisal is the estimated cost of irrigation works. The question of the tangible, physical, and water values back of the bonds always arises in the event of financing. In the case of a new district, these are largely estimates and generally involve the taking up of new rights. Their value at the time of filing is almost nil, so that the value of a proposed project is generally limited to the physical works, to the rights of way, including interest during construction, and to organization expense, etc.

On some of the projects with which the writer has been connected, old water rights were purchased, including old reservoirs, reservoir sites, and old canals. In such cases appraisals were required for the works purchased, particularly for the value of the water itself.

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* Presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927. The author suggests that the discussions by M. C. Hinderlider, R. I. Meeker, Lynn Crandall, and E. B. Debler, Members, Am. Soc. C. E., on the Symposium on Administrative Water Problems (which follows his paper), be read in connection therewith in order to clarify the laws and practices of the Arid States on this subject.

† Denver, Colo.

The new works generally destroyed or replaced old ones, the water being used over the entire new system. At the time of purchasing old works, and primarily to obtain the old water rights, whatever amount was paid for those portions of the old works that were abandoned or destroyed, was added to the cost of the water or water right.

Early Appraisals

The writer's early appraisals were based almost wholly on data gathered concerning the actual sale and transfer of water, a rather simple matter. Furthermore, it was found that there was a general impression in every locality as to how much water was worth. This influenced the appraisal. These opinions included, not only the value of the water right itself, but of a proportionate interest in the works. On some canals, construction costs are far more than the value of the water itself. On other canals, the physical works are simple and of very low cost. Hence, it must be borne in mind in speaking of the value of water that the physical values of the works and of the water itself must be separated.

Similarity in Values

The market value of an acre-foot of water in a given locality does not vary whether its use be domestic, manufacturing, or agricultural. Further, stored water has little more value than unstored water. A proviso, however, is essential to this last statement, namely, that the use of the unstored water must be economical and beneficial.

Consider in the abstract an acre-foot of water. It may be easily obtainable during the flood season, yet in the absence of other acre-feet, procurable at other times and necessary to bring the crop into the flood period and later carry it to full maturity, that acre-foot of flood water would be without value. If, however, it entered into the production, in whole or in part, of a crop between seeding and harvest, it has a value almost, or quite, equal to the value of an acre-foot used at any period of crop growth. The use of flood waters means, generally, a saving of the stored supply. The law of supply and demand alters the prices, of course; but intrinsically, and on the basis of crop production, the value of flood water is as great as that of water obtainable during the low flow period.

The conclusion as to the uniformity in value of water may be startling to some engineers. Except for the need of water for agriculture in the semi-arid region, there would be enough for domestic needs and for manufacturing. The water used for agriculture can be purchased and converted to a domestic use, it can be condemned and appraised at its agricultural value. It is agriculture, therefore, causing, as it does, a demand in excess of the supply, that gives value to water. It is a more largely consumptive use than either of the other two, although in both manufacturing and domestic supplies, the consumptive use is much greater than is generally believed.

The writer has investigated consumptive use by the City of Denver, Colo., and has concluded that during the summer months about 66% of the gross diversions was returned, while in the winter the quantity was 80 per cent. As regards the consumptive use of water for manufacturing purposes, principally for cooling, about 10% of the supply was lost in transit through the power houses and mills. The idea that practically all the domestic and manufacturing supply returns to the river is just one of those dreams that does not come true.

The value of water for domestic purposes should not be based on the city's necessities, nor on what the city could afford to pay, but rather on the value of water for agricultural purposes. As to manufacturing, the great intrinsic value lies in the water used for condensation purposes. Large generating plants are often forced to go to localities where water is available rather than to build on sites in all respects preferable, except that the agricultural interferes with the manufacturing use. If the water is used for the direct generation of power the value could be determined by comparison with the cost by steam or the use of electric power.

The statement that an acre-foot of water, whether stored or direct flow, varies little in value, is not so obvious as the first conclusion. It is true that an acre-foot of water in the latter part of the season is much more valuable than in the early and flood periods, and that an available quantity of water during the time of very low flow is much more valuable than during the flood period. This is confirmed by the variations of value of water having different dates of decree.

Obviously, flood rights are of considerable intrinsic value. They permit a greater area to be cultivated, encourage the growing of early maturing crops, and stimulate a more general rotation of crops than would otherwise be practiced. Reservoirs built for the storage of water are generally much more expensive per acre-foot of capacity than are canals. Deducting the cost of structures from the value of water plus structures would show that water values are more nearly equal than the values of structures plus water. In the two types the cost of a dam with its accessories often approaches the value of water plus structures even after it is fully used and is a successful and going concern. Thus, waters having decrees for storage purposes and for direct use have values varying with the dates of decree and independent of the method or character of works used in securing it.

In his early experience in appraising water rights, it seemed to the writer to be simply a matter of ascertaining the sale value of the water as determined by a number of sales under comparable conditions and in the same locality. The use of such data, however, led to the most irreconcilable differences as well as to absurd results. It soon became apparent that there was a vast difference in value between water "direct-flow" as compared with "stored water". In some cases, the opposition engineers insisted on converting one into the other, and basing the value on the quantity theoretically procurable during the entire irrigation season. While such a method appeared ridiculous at that time—and, indeed, the results were ridiculous—yet it now appears that it is, in part, correct. Due regard, however, must be given to the quantity actually produced by each and to the date of the decree. To judge stored water by comparing it with value of water diverted under old decrees, or *vice versa,* although the time of use and application were the same, would be wrong. The cost of structures in each case added to the water value gives that generally quoted.

APPARENT AND TRUE VALUES

To those familiar with the transfer of water in Colorado from one ditch to another, the purchase price is not a true measure of the value. In most cases there are conditions and provisions in the agreement which very materially alter the given value. The purchase of 10 sec-ft. of water at $3 000 per sec-ft. is $30 000, but when to this figure are added the Court costs of several thousand dollars, and the cutting down of the quantity allowed to be transferred, the rate might be, and usually is, more than doubled. Where water rights are purchased and transferred, it is almost invariably true that the seller is anxious to sell and the purchaser is not compelled to buy; that whatever that seller gets for his surplus water is so much net money to him. Too frequently, he is selling something he has never used or, at best, something that he at one time used but through changed conditions, due to seepage returns, etc., he no longer needs.

THE METHOD PROPOSED

In an effort to reconcile these differences, the writer sought a more rational method for the determination of the value of a water right. In one case, he selected three of the larger, more substantial, and well-known ditches on the Arkansas River—ditches on which not only were the values of the structures known, but for which the daily flow into the canals over a period of at least eight years was of record. As each of the three canals has a number of priorities of different dates, varying not only as to their own rights, but as to those of the other two ditches, the appraiser was able to construct curves showing value and priority dates.

In this case about 200 priorities and more than 100 diversions were to be appraised. The uses were: Domestic, the year round; manufacturing, some used intermittently and some continuously; and, agricultural, where the irrigation season varied from 60 to 300 days in the year. Also, the records for the domestic and some of the manufacturing uses were expressed in gallons per year without reference to the maximum flow or the hours or days when the pumps were in operation. Therefore, it was necessary to develop curves showing the value of different priorities and the percentage of their efficiency as compared with a 365-day use. It was also necessary to use total volumes diverted during the year, expressed in acre-feet, rather than the irrigation season and the rate of flow expressed in cubic feet per second.

It was decided, therefore, to use as the maximum possible, the theoretical quantity that could be diverted under the decree, and to determine the percentage of efficiency or delivery by comparing the actual delivery with this theoretical maximum.

First, the selling price of a share in the ditch company was ascertained. For each ditch the price had been quite uniform over a period of four or five years, and several transfers of comparatively large blocks of stock had been made. Furthermore, whether the stock was sold in quantity or in individual shares, the price varied little. The value of the works, including the water rights, was determined by multiplying the average sale price by the number of shares. From the books of the companies and from personal investigations on the ground, the writer determined the replacement value of the structures, making due allowance for the fact that some parts, embankments in particular, were more substantial and valuable than when they were first constructed, having passed safely through the most dangerous period of their existence, namely, the first few years. Having determined the structural and going-concern value, as well as the full value of the works, the difference between the value of the stock and the value of such works was called the value of the water rights.

In reality, the three ditch systems are mutual ditch companies in which the water users own non-interest bearing shares roughly proportioned to the area irrigated, or to the necessity of the users. Generally speaking, stock is sold only to users, and the proceeds are used for the payment of debts, for improvement, or for the reduction of assessments. The ownership of shares without land on which to use the water would simply mean annual assessments without benefit.

Next, the quantity of water carried by the three canals was ascertained over a period of eight years. A longer period would have been desirable, but the records were not uniform beyond that period; indeed, many were missing, so that, perforce, study was limited to eight years. Having ascertained the average number of acre-feet of water diverted per annum, the value per acre-foot was determined by dividing the water-right value by the number of acre-feet diverted. An example, taken from the report, will show clearly the method used:

"FORT LYON CANAL

"Priorities are April 15, 1884 for 164.64 cu. ft.
March 1, 1887 " 597.15 " "
1893 " 171.20 " " "

The value of this system is, according to its President, $6 000 000, and that of the structures about $2 000 000, leaving the water value as $4 000 000, with $600 000 of indebtedness, a total value of $4 600 000.

A careful analysis of the daily records for eight years shows the flow credited to the respective priorities to be as follows:

From reservoirs	47 000	acre-ft.
Average for the 1884 decree..................	80 000	" "
" " " 1887 "	90 000	" "
" " " 1893 "	8 300	" "
Total.............	225 300	acre-ft.

The value of the water per acre-foot is, therefore....... $20.42

Reservoir water = 47 000 × $20.42 = $960 000.
Decree, 1884, therefore, is worth $1 635 000 = $9 925 per cu. ft. per sec.
" , 1887, " " " 1 838 000 = 3 080 " " " " "
" , 1892, " " " 169 000 = 990 " " " " "

The 1884 decree delivers 80 000 acre-ft. out of a possible 119 200, for 365 days during the year. Its percentage of efficiency is, therefore, 67.15. Therefore, $9 925 represents 67.15% of the maximum value; that is, 100% would be worth $14 900 per cu. ft. per sec. Similarly, the other priorities give the same result: The 1887 decrees have an efficiency of 21%, and the 1893 decrees, one of 6.7%, for this particular canal. In case the value of the physical works was placed too high, the value of the priorities would, of course, be greater than that shown.

In ascertaining the number of acre-feet delivered under each of the decrees on the canals, the daily records were studied, and whenever only the oldest decree was being filled or partly filled, the entire quantity was assigned to the first decree. When a quantity was shown in excess of the first decree, but not greater than the first and second combined, the remainder was assigned to the second decree; whenever the quantity carried was in excess of the two first decrees, the excess was credited to the third decree; and so on through all the decrees. The result of this study was that the writer had the average number of acre-feet delivered per annum by each decree, and multiplying that quantity by the acre-foot value as determined gave the value of the decree. Dividing this by the quantity of the decree, in second-feet, gave the value per cubic foot per second. As the values were to be used in comparison with supplies for domestic use, for manufacturing power, and for irrigation, and with stored and unstored water, it appeared necessary to have some common basis for comparison. The quantities diverted under each priority varied in amount from day to day and from year to year. The only common ground seemed to be what would have been the quantity diverted had every priority run for the entire year.

Grouping Priorities

Having ascertained by this method the value of ten or twelve priorities on the river, it is possible to construct a curve that can be used in determining values of other decrees. In the case used as an illustration it was shown that all priorities having a date earlier than 1874 were of equal value. That this was true was demonstrated by plotting all the priorities as to date and quantity, the quantities being the aggregate of the water decreed prior to a certain date. In this way, it was ascertained that even in years of minimum flow the supply in the river was in excess of all the decrees of 1874 or earlier. It was found, also, that priorities could be grouped. These groups were separated by some decree calling for a very large quantity of water. Thus, the application of the data and the determination of the value of specific water rights were much simplified.

In Fig. 1 the curve to the left relates in part to the three canals investigated and to their actual value as determined, while the several curves to the right represent values as determined from the sale of water to be transferred, illustrating the impracticability of using that method for the determination of real values. These latter curves illustrate, also, the gradual increase in values of water rights over a period of years.

Starting at zero on the year of appropriation, they rise uniformly to the time when the first information is available, about 1897; from that point the actual sale prices indicate quite a uniform advance in values, a sudden rise during the World War period, with no rise, possibly a decrease, subsequent to about 1921. The steepness of the curve between 1900 and 1905 is probably due to the effect of the Reclamation Act, while the break in the curve in 1907 is probably due to the financial panic of that year. However, one is not justified in using this curve as anything other than illustrative.

It should be clearly understood that this method is suggested for the purpose of laying a foundation only, for detail study. On each stream the value must be determined independently of that on any other system, but even after having obtained a basis for further detailed work, it should be used more as a guide and to permit the construction of curves where many data are missing.

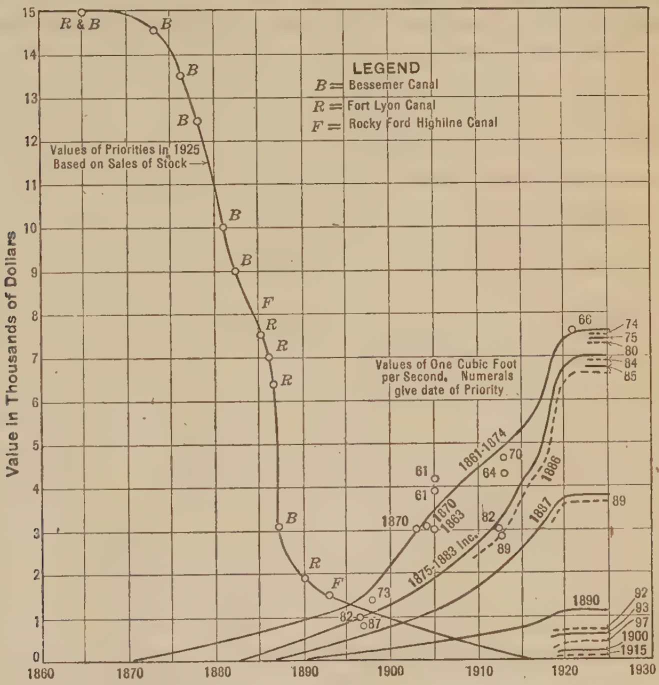

FIG. 1.

The law of supply and demand rules, and it often happens that under an old priority, where there is an abundance of water, the value of the water is less than under a later priority, where the abundance of land makes a marked demand. The matter of crops, productivity of the soil, the markets, all these must be taken into consideration by the appraiser in his detailed study. In arid and semi-arid regions the value of the water is its value for its most important use, namely, for irrigation. Therefore, the appraiser may determine from curves based on agricultural use the value of the domestic water rights for cities, and the value of water for railroad purposes. In the latter case, however, it often happens that the value of the water is the cost of bringing in some substitute which might involve hauling for long distances. Where water is as scarce as this, however, a value could hardly be established under the method suggested.

Colorado is fortunate in having a law which requires the installation of automatic registering devices on all canals. Since that law was passed the State Engineer has exercised diligence in having automatic registers placed throughout the State. When the records from these are matured and available, it will be an easy matter to ascertain the stock value of an irrigation system and the average number of acre-feet diverted per annum and from this to deduce the value of an acre-foot of water. It will involve no great amount of labor to place the proper values on water rights. As has been stated, this can be done safely only by those who are familiar with the particular locality or willing to devote a large amount of time to ascertain local conditions.

DISCUSSION

JOSEPH JACOBS,* M. AM. SOC. C. E. (by letter).—This paper is of value and interest because it deals with a subject that grows in importance each year, in accordance with a steadily expanding population, agriculture, and manufacturing industry and their persistent demands for additional water supplies. It seems to deal, not with the broad, general subject of water-right valuation, as its title implies, but rather with a restricted phase of it which has to do more particularly with the relative values of several irrigation water rights in the same mutual ditch system, but under decreed differences as to priorities and quantities of water.

It is frequently contended, in the valuation of public utility properties, that no allowance should be made for water rights, because the actual ownership of the water rests with the public, and there have been many findings and rulings by public service commissions sustaining this view. As against allowing nothing for the water right, a more equitable showing could be made for the principle of dividing the water-right value between the public and the utility owner, in cases where the water-right value is based on consideration of the extra cost involved in developing the next best available water supply. This would be based on the theory that the consumer public would otherwise lose, completely, the money value of its natural advantage of location with respect to the cheaper supply. That water rights do have value, however, and that this shall be recognized as a principle in public utility valuations, unless there is definite stipulation to the contrary, has been too often attested in decisions of the higher Courts to be seriously questioned. In the recent Indianapolis Water Case the United States Supreme Court stated that "the water rights must be included".

Further evidence that owners attach value to their water rights inheres in the history of the violent and costly struggles and encounters, both physical and judicial, that have been engaged in to establish and maintain such rights.

The Federal Power Commission also, in a way, recognizes this principle of water-right value. In its published rules and regulations which provide (in respect to power developments on Government land within its jurisdiction) that the licensee shall pay an annual fee, based on the installed capacity of the plant up to the power capacity of the site, ranging from 5 cents per h. p. for the first year of operation to 25 cents per h. p. for the third and subsequent years of operation. These figures have definite capitalized values, depending on the assumed earning capacity of money and, with a known power head for any given case, such values could be expressed in terms of dollars per second-foot of water. While it is believed that thus basing the charge on plant capacity may indicate recognition, by the Commission, of the principle of water-right value, the charge is not, of course, a measure of such value. In the first place, the Commission states that the charge is "for reimbursing the United States for the cost of the administration of the act." In the second place, the charge may be considered as being in part for rental of land occupied by the project and, if the available water supply is in excess of any possible power-market demand, it may be regarded as attaching more to the power franchise than to the water right, because such a distinction can be made.

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It is less difficult to prove that water rights have a value that should be recognized, than it is to determine what that value is. The valuation method to be used, and the final value to be determined, depend in part on the purpose of the valuation; in part on whether a private industry or a public utility is being dealt with; and, finally, on the elements that make up the value of the water right. It is not possible to formulate a simple definition of what these elements are because of the many qualifications and exceptions that have application to specific cases. Moreover, by reason of statutory differences, what may be valid in one State, respecting water rights and their valuation, may not hold in another State. There are numerous factors that have a bearing on water-right valuation. Some of the more important of these are:

(*a*) The State laws and regulations on which the validity of the water right rests.

(*b*) The priority, amount, and duration of the water right as defined by Court decree or as affected by franchise provision. This involves also the question of storage rights as distinguished from normal-flowage rights.

(*c*) The franchise, and the limitations thereof, under which is operated the project to which the water right attaches. A short-lived franchise would detract from the value of a water right and the franchise may even stipulate that no value shall attach to the water right under certain conditions.

(*d*) The amount that was legitimately expended, by purchase or otherwise, in securing the water right.

(*e*) The value, for agriculture or other ordinary purposes, of the land occupied in the development of the water right as, for instance, the land required for dam, power house, canal head-works, etc.

(*f*) Whether or not the project to which the water right attaches is fully or only partly developed and whether the market for the product that the project supplies is immediate or only prospective and whether it is permanent.

(*g*) The abundance of the water supply. A supply in excess of present and prospective market requirements would tend to depreciate, as would a restricted supply tend to appreciate, the unit value of a water right.

(*h*) The cost of the next best available water supply or other adequate substitute.

(*i*) A possible higher-valued alternative use for the water.

(*j*) The capitalized value of net earnings of the project above operating expense and normal interest return on the investment in the physical plant.

The bearing of most of these considerations on the value of a water right is too obvious to require comment, but a brief further discussion of some of them may be desirable. Referring to Item (*d*), it would seem that while the amount that was legitimately expended in securing the water right would not necessarily fix its maximum value, it would normally fix the minimum value that should be allowed. The word, "normally," is used because if the conditions surrounding the business were such that it was impossible to earn a fair return, regardless of rates, and such instances can be cited, then the water right might have no value at all. The Item (*e*) consideration affords another criterion of value. If the value, for ordinary purposes, of the land required to utilize the water right, is deducted from the actual price paid for the land, or if it is deducted from the value of the land for ordinary purposes multiplied by the average public utility factor that obtains in the district, in connection with real estate purchases, then the remainder may be regarded as one measure of the value of the water right. This, however, is not deemed to be a rational or dependable method of water-right valuation because of the uncertainties and the fortuitous circumstances that affect the "public utility factor". In many instances the land has no value at all for ordinary purposes.

In respect to Item (*f*) it may be said that the status of the project, as to the extent of its development and the character of its market, must be reflected in the value of the water right, although it is believed that these factors would be more fully and more properly reflected in "development cost" and "going concern value", if the intangible elements of value in a property were segregated instead of being combined into a single item, as is so frequently done. While a developed project with good markets for its product would tend to give substantial value to a water right, no Court would be inclined to concede a material present value to such right on account of a prospective future market unless that market was definitely assured and reasonably close in point of time.

In connection with Item (*g*) which indicates that the magnitude of the water supply, in relation to demand, affects the water-right value, a distinction between intrinsic value and commercial or market value must be recognized. In the semi-arid States, for instance, irrigation water has intrinsic value because agriculture would be impossible without it, but, even if the results of this agriculture are highly profitable, it does not necessarily signify that the water right is of high commercial value. If the available water supply is in excess of possible demands, and is obtainable from the State without cost other than nominal filing fees, then, manifestly, the water right proper could have no great commercial value, for no one would pay a high price for that which he can obtain practically for nothing. Such value as was warranted by the net profits from the agriculture served, would be reflected mostly in the land and, according to the extent that it was reflected in the stock of the ditch company, the value of the latter, in excess of that represented by physical plant investment and development costs, would more properly attach to "going concern value" and to "franchise value" than to water-right value. If, however, there was not an abundance of water for future or other requirements then the water right would properly absorb a greater part of the excess value previously mentioned.

The cost of the next best available water supply or other adequate substitute and the possible higher valued alternative use of the water, referred to in Items (*h*) and (*i*), not only afford distinct bases for determining the value of a water right, but they may, in certain cases, become the controlling considerations. Certainly, they would have application in dealing with water supplies for private industries and, in connection with public utility properties, they would, ordinarily at least, fix the maximum allowance that could be made for the water-right value. If the water right may appreciate in value, or attain maximum value, from these considerations, a logical corollary would be that if in the future, through changed conditions in stream run-off, or through new scientific developments, a substitute may be provided at lesser cost, then the value of the old plant would depreciate and this loss in value would also, in part, be reflected in the water right.

The most fundamental consideration, the one that has the most general application, is that defined in Item (*j*). The capitalization of the excess net earnings mentioned, measures not the value of the water right alone, but of all those elements classed as "intangibles". If there is no such excess, actual or prospective, then, from an investment standpoint, no value could be assigned to the water rights or other intangibles, and the allowed value for a property would need to be assigned wholly to its tangible elements. Intangibles are frequently lumped in a single item because that may suffice for the purpose in hand, but for any refined valuation they should be segregated, and particularly so, when dealing with the specific inquiry of the value of a water right or any other single intangible. This lumping of intangibles tends to confusion, and not infrequently this lump sum is expressed wholly as a water-right value when that may not be the case at all. The intangibles may include "development costs", "going concern value", franchise value, water-right value, etc. The first two are things apart and rather definitely determinable. Franchise value and water-right value, however, may sometimes be confused, but each individual valuation case will usually afford some rational basis for segregation.

As applied to private industry, the logic of capitalizing excess net earnings as a basis for determining water-right values can hardly be questioned. It may also be correct as applied to a public utility, depending on the terms of the franchise under which the utility is operating, and, particularly, if the valuation case is one in which the rates and earnings of the property are found to be fairly definitely established, regardless of the method by which those rates may have been reached. This, in a sense, begs the question in respect to those public utility properties for which, in theory at least, the rates depend on the rate base rather than the reverse. For rate-making purposes, in such cases, a regulatory commission would have to use some independent method of arriving at the water-right value.

All these considerations and methods of determining water-right values may have application in certain cases yet none of them has universal application to the exclusion of the others. They may lead to quite different results and the one that should control depends on local conditions and on all the circumstances surrounding the particular case in hand. There is too much tendency on the part of appraisal engineers to adhere to rigid values in valuation work, some adopting that method which always yields a high valuation and others that which always makes for a low valuation. There are, of course, certain fundamental principles that must be observed, but an important principle that deserves broader recognition than it usually receives, one that will tend to dispel the prejudicial view, is this: That no single method of valuation can be rigidly applied to all cases, that each case is a thing apart, and that the valuation method, to be applied, must depend on, and be adapted to, all the circumstances and conditions surrounding the particular case and business involved. Valuation work is complex at best, and to arrive at just results the appraisal engineer must possess, in addition to his technical knowledge, a discriminating fairness and an abundance of good judgment and common sense.

The author states that in building new irrigation works to replace old ones, the amount paid for those parts of the old works that were abandoned or destroyed, was, of necessity, added to the cost of the water or the water right. This, it seems, is properly a development cost and in no sense a proper charge against the water right. That it would add no value to the water right proper is evidenced by the fact that no one would pay this extra cost for the water right alone if, for instance, there was an abundance of water in the stream. This development cost would be reflected, of course, in the price placed on the stock of the company, but it would not, in the writer's judgment, represent water-right value.

The author also states that in his early appraisals it was deemed merely necessary to ascertain the prevailing sale value of water. When conditions are stable the market or sale price of water stock is at least a criterion of value that should be considered and in some cases it may be important. There are cases, however, where the market price of water stock is largely fictitious, bearing but little relationship to earning capacity or to actual investment. In such cases it would not be safe to deduce a water-right value from the market price of the water stock. The water-right value must, of course, be segregated from the all-inclusive stock value. At another point, the author himself recognizes the unreliability of this market price method of determining value.

The author's statement that stored water has little more value than unstored water requires some qualification. The truth of it depends much upon the flow characteristics of the stream involved. If the unstored or normal flowage water is exhausted during the very early months of the irrigation season, as it frequently is, then, even for identical volumes, the stored water would be more valuable to the crop than the normal flowage. In general, it is believed that stored water is more valuable, volume for volume, than unstored water, because it is under definite control, it can be delivered as, and when, required, and it is available in periods of the most acute need. The comparative values of like volumes of stored and unstored water, however, may be something quite different from the comparative values of a water right from normal flowage during the irrigation season and a water right for storage during the non-irrigation season. The exact definition of the water right and the stream characteristics are all important, because the average number of acre-feet of usable water that each water right will produce annually must be determined before a comparison of values can be made. The term "usable water" is emphasized because, not only does the right usually cease when beneficial use ceases, but a water right takes on value only as it has a dependable water supply, the equipment to deliver the water, and a market to absorb it. In water-power development, and also in domestic water supplies, the rate of stream discharge rather than the total volume of run-off may often be the more important consideration, so here, again, its adaptability to such a condition, makes stored water more valuable than unstored water.

Where there are several water rights on the same stream the priorities of those rights, and their time limitations within the year, if any, are important elements affecting value, and the length of the stream-flow record may also appreciably affect the water-right value. The writer agrees with the author that a longer record than eight years is desirable. A short-time record cannot develop true stream characteristics and a long-time record is likely to reveal worse low-water conditions than the shorter record would show. The effect of this would be to give greater relative value to the early priorities, and it may also give a greater value per unit volume of water. The length and dependability of the available discharge records are elements that should not be disregarded in water-right valuation work.

The author, in his development of water-right values for several specific cases of canal systems, does not indicate how the gross valuation of these properties was initially determined. He starts with an assumed, definite gross value, which is apparently the market value, and an assumed value for the physical elements of the property. Calling the difference between the gross value and the structural value, the value of the water rights, he develops a method of apportioning this latter value among the several water rights according to their priorities. The writer would first indicate that, since the water-right value results directly from the relative gross and structural values, the important primary consideration is the correctness of the method by which those latter values were determined. The author does not indicate what these methods were. This difference between gross and structural values is not necessarily the water-right value alone. It is the aggregate value of all the intangibles, and these may include elements other than water rights.

In the Fort Lyon Canal case it is stated that the gross value of the system is $6 000 000, the structural value, $2 000 000, and that the water-right value is, therefore, $4 000 000 which, with an indebtedness of $600 000, gives a total water-right value of $4 600 000. It is not clear why the $600 000 should be added to the water-right value. If it represents value at all it is presumably included in the gross value of $6 000 000, and if it represents accrued operating losses that are in fact "development costs" they are not properly a water-right increment of value.

The author's method of segregating the total water-right value among the several rights on the basis of their priorities, is thought to be the logical and probably the only rational method for such segregation. It is believed, however, that in determining the quantity of water deliverable during the year under any decree, the possible yield for the full year should not be taken, but only the yield of that portion of the year during which the water can be put to beneficial use; as, for instance, during the irrigation season only. Unless the decree specifically permits it, and there are facilities to deliver, and a market to absorb, the water deliverable during the non-irrigation season, the normal-flow water right for that period would have no value. Even storage rights for that season might have no value if the cost of the storage works should be greater than the value of the water stored. A recasting of the author's figures, on the basis of water yield during the irrigation season, will give an entirely different set of water-right values.

Determining the so-called efficiencies of the various decrees as bases for determining the value of a water right of 100% efficiency would seem to be an unnecessary elaboration because, after the value of the acre-foot of water is determined, one need only multiply that figure by 724 to obtain the theoretical value of a continuous flow of 1 sec-ft. The word, theoretical, is used advisedly because no such 100% efficiency can obtain, regardless of the provisions of the decree, unless the water is continuously available and there exists facilities for its delivery and a market to absorb it in beneficial use. It is possible that, with storage and the development of markets, these conditions may sometimes be met, but without these essentials satisfied, such figures as the author develops for the value of water rights of 100% efficiency are academic and likely to be misleading.

FRED H. TIBBETTS,* M. AM. SOC. C. E. (by letter).—To the present the engineer attempting to make a rational valuation of a water right finds little precedent and practically no authoritative literature to guide him. Mr. Field, accordingly, deserves all the abundant credit due for his important pioneer effort. The paper is of the greatest interest and value, containing a clear, concise, and logical method of fixing the valuation of a water right where the data required can be readily, and with certainty, procured.

At a time like the present, when agricultural securities are notably below par, and particularly in California, where State irrigation districts have issued more than $100 000 000 worth of bonds, it would be of material assistance in financing additional developments if the general public, bankers, and Courts, could be educated to a proper recognition of the true value of reliable water rights available with certainty for irrigation development. It is the writer's hope that this paper will assist materially in creating recognized methods of fixing substantial values for water rights.

Mr. Field's method is to obtain the total value of a public utility, or mutual water company, furnishing water for irrigation, by determining the market price of its total outstanding stock and presumably other securities. From the value thus obtained he deducts the appraised value of the physical properties and assigns the remaining value, if any, to the water right. The few public utilities serving water in California, seldom have a total stock value equal to that of their appraised physical properties. Hence, although it is recognized that their water rights have substantial values, Mr. Field's method of obtaining such values is quite inapplicable. As a broad generalization, it seems highly conjectural to assume that all the stock value in excess of the reproduction costs (less depreciation) of physical properties, represents the value of the water right. This residual must also include the sum of all "intangible values", such as promotion and development expenses, organization, "going concern", etc. The total stock value of any corporation is also measurably dependent on public confidence in the officers of the company, and many times on the efficiency of its advertising. There are many corporations in which at times the total market value of the stock may bear but little relationship to the value of the physical properties. A good example, in California, is a certain boom oil company luridly advertised and over-capitalized, in which over-issued stock with no valuation whatever of physical properties back of it, may exceed $50 000 000 to $100 000 000.

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The writer is somewhat of the opinion that Mr. Field's method attacks the problem from the wrong end. He is inclined to the belief that if he were to purchase a large amount of stock in the water companies which Mr. Field discusses, he would first appraise the physical properties and then, from independent considerations, would appraise the water rights, and, finally, from the total of these would determine what he considered to be the value of the stock. This procedure seems more logical than to accept public opinion regarding the market value of the stock as conclusive, and from this to assert that the difference between this market value and the value of the physical properties must represent the value of the water rights.

The writer is in fundamental disagreement with the author when he concludes that the value of an acre-foot of water is the same, whether it is used for domestic purposes, or for those of manufacturing and agriculture. It is certain that in California the cities can and do pay far more for domestic water than the farmer can pay for irrigation water. Mr. Field's statement that "except for the need of water for agriculture in the semi-arid region, there would be enough for domestic needs and for manufacturing", seems quite inapplicable to regions like Southern California, for example, where the duty of water per acre for urban development is about the same as for agricultural development, and where large quantities of water must be imported from far distant points to satisfy the domestic and industrial needs alone. The writer sees no reason why the value of water for domestic purposes should not be based on the city's necessities and on what the city can afford to pay; in particular, he sees no connection between the value of water for domestic and for agricultural purposes, and can discover no reason why the value for domestic purposes should be based on that for agricultural purposes.

Mr. Field suggests that the idea of converting the cubic feet per second, into acre-feet and basing the value of a water right on the number of acre-feet thus produced, at one time "appeared ridiculous". The writer is entirely in agreement with Mr. Field, however, in that this is probably the correct method. As a matter of fact, this is the only method for valuating a water right that the writer has ever used or considered. In determining the value of water rights for irrigation purposes the "useful yield" during the irrigation season should be determined in acre-feet, and the valuation based on this unit. The "efficiency" of a water right for irrigation purposes should then be the ratio between the number of useful acre-feet produced by this water right during the irrigation season and the number of acre-feet during the irrigation season (included in the irrigation project's demand curve) which would have been produced by a second-foot flowing continuously during the peak demand of the irrigation project, and at other portions of the season reduced in proportion to the project's reduced demand.

While the author's method is logical and excellent for the conditions outlined, the writer believes that recent appraisals of water rights on the San Joaquin River, in California, were based on broader and more general principles.

The problem presented was unusually complex and complete, because nearly all conceivable elements of value were present in substantial quantity. The important water rights on a major stream controlled by a single private interest were proposed for transfer outright to a newly organized water storage district. As it was conceded that the district would not attempt to condemn the water rights, the problem was not complicated by intricate and archaic legal theory, as is so frequently the case. The appraisals described herein led to a final valuation of about $9 500 000. This was a compromise figure agreed on by the writer, representing the owners of the water rights, and A. Kempkey, M. Am. Soc. C. E., Consulting Engineer for the District. The theories herein described are those developed jointly by Mr. Kempkey and the writer.

Chief consideration was given to the following elements:

1.—The market value as established or indicated by actual transfers of comparable rights under conditions similar to the one considered.

2.—The substitutional service value, or the cost of securing by some other method the same water supply as is obtainable by the appraised water right. Two other methods were readily available in whole or in part: (*a*) Impounding flood water; and (*b*) pumping ground-water.

3.—The water value, or the capitalized sales (per acre-foot), outside the District, of irrigation water at diversion points, under similar conditions.

4.—The land increment value, or, the net increase in value of the land to be irrigated, caused by the acquisition of a water supply for irrigation.

5.—The land decrement value, or the decrease in value of lands irrigated or partly irrigated under the appraised water right, arising from depriving them of their water right. This is an unusual condition, but it obtained in the proposed transfer, thus rounding out the problem. Water was to be taken from 178 000 acres of "grass lands" of inferior quality, irrigated for pasture, and transferred to new lands as yet unirrigated and arid, but of superior quality.

1.—Market Value.—It appears to be the general legal opinion that "market value" as shown by the sales of other water rights of a similar character under essentially analogous conditions, constitutes the true criterion for a valuation of a water right. Like most legal formulas, this opinion is theoretical and evasive. The "market value" of wheat may be thus determined. Water rights cannot be standardized; no two are alike; there are few *bona fide* sales in which the price paid for the water right can be clearly identified; and in the last analysis final conclusions are subject entirely to matters of individual, professional, and expert opinion, as to the degree of comparability of the water right appraised with other water rights the market value of which has been estimated from purported sales.

It would also seem obvious that if there were sufficient sales to establish a "market" for water rights, then the value thus established must take into consideration all the usual elements of supply and demand in ordinary commercial transactions. In particular, it must be chiefly affected by the value of the water right to the purchaser, and its value to the seller. These two values must, in time, reflect chiefly the values determined by Elements 2 to 4, and particularly the last two, Elements 3 and 4.

Included in voluminous expert testimony in the Spring Valley Water Company rate cases in San Francisco, Calif., are deductions for the market value of irrigation water obtained by this method and varying from $70 200 to $1 272 per sec-ft. It will be obvious that the results obtained are entirely a matter of professional judgment as to the degree of comparability of the different water rights.

2.—Substitutional Service Value.—It is not always possible to obtain any substitutional service value because frequently no source of water supply can be obtained other than the one appraised. The writer believes, however, that much weight should be given to this matter whenever it can be clearly shown that there are one or more possible sources of water supply other than the one appraised. In many cases reservoir construction for the storage of flood water will furnish an entirely satisfactory, substitutional source of supply. Opinions may differ as to the relative value per acre-foot of irrigation water thus obtained. The writer believes that irrigation water obtained from a natural stream flow, the yield of which is known from long and reliable records, can be estimated more accurately, and hence, is more valuable, than a corresponding quantity of water which is merely the estimated yield from a storage reservoir not yet built. Mr. Kempkey, however, held that reservoir releases per acre-foot are more valuable than corresponding natural stream flow, because the construction of the reservoir allows, in the future, much greater elasticity of operation and variation in reservoir releases with variable seasonal weather, and variable crop production, over a long period of years.

Substitutional service in large part may also be obtained from wells, but here, again, the cost of obtaining a well supply, as yet unconstructed, cannot be estimated over a long period of years with any great confidence, because of the probable variation in pumping head and in cost of power.

3.—Water Value.—It is generally much easier to obtain reliable records of the sale of natural stream flow for irrigation ditch diversion, or of reservoir releases for irrigation purposes, than similar values of the sale of water rights. However, it may still be a matter of opinion as to the degree of comparability of the water sold, with that to be furnished under the right appraised. When reliable figures for the market value per acre-foot of irrigation water are thus obtained, they can be capitalized and used as a measure of the value of similar quantities of water to be obtained under the rights appraised.

4.—Land Increment Value.—Increase in the net value of irrigated over unirrigated land can usually be determined with considerable confidence, providing irrigation has been long practiced in the district under consideration. In obtaining the net increase in value of irrigated lands because of the irrigation water rights, the difference in value between the irrigated and the adjacent unirrigated land without water rights should be obtained, and from the difference thus obtained, must be taken the cost of improving the land, leveling it, checking it, constructing irrigation distribution, etc. Not all the net increase in value, as thus obtained, can be claimed for the water right; a portion should be claimed by the land and a portion by the agency which has provided and operated the physical works for the diversion and conveyance of the water.

In this connection it may be emphasized that at the instant the physical works for diversion and conveyance have been completed and the land owner has prepared his land for irrigation (with no water having been actually diverted), there does not exist a completed and valuable right in the sense that such a right will exist after many years of diversion and use. It may be assumed, therefore, with propriety that the water agency on the one hand and the consumer on the other, after years of operation, have contributed jointly (through diversion in one case and use in the other) to the creation of a property right having a very substantial value, to wit, a perfected water right. It will also appear that the contribution in each case has been independent of investment, and is based solely on the physical fact that the water agency, on the one hand, has diverted the water, and the consumer, on the other hand, has used it. Obviously, diversion by the water agency would be of no value without use by the consumer and in the case of the particular land in question, use could not have been effected without diversion by the public utility. It appeared, therefore, equitable that each should share equally in the property value thus jointly created.

5.—Land Decrement Value.—This method is clearly inapplicable, except as to lands that propose to give up their water right, resulting in their being dried up and rendered less productive. In the case considered, the value to the owner (of the dried-up lands and of the water right) was obtained by estimating the difference in value of the land, with and without the irrigation to which it had been accustomed, or by capitalizing the difference in revenue or rental from the same lands, with and without their accustomed water supply.

Appraisal Units.—To obtain a quantitative expression of the valuation of the different water rights the "safe net yield", in acre-feet, during the assumed irrigation season, was determined for each water right. The "safe net yield" was a term applied to the estimated minimum quantity of water allowable in the minimum or dryest year. The normal water supply at the diversion points in the entire district of 554 300 acres was estimated at 2.45 acre-ft. per annum, with a monthly distribution as shown by Fig. 2 and Table 1. It was assumed that shortages in minimum years were allowable up to $33\frac{1}{3}\%$ of the normal supply. Fig. 2 shows the assumed water demand of the district.

A hydrograph, based on long years of record of the actual yield of the principal water right appraised, is superimposed. Under these conditions a theoretically perfect water right, that is perfect in monthly distribution, would yield 285.5 acre-ft. of useful water per sec-ft. during the irrigation season.

TABLE 1.—AVERAGE ACTUAL YIELD OF THE WATER RIGHT.

Month.		January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.
District demand ...	Percentage per month..	0	0	6	12	21	20	17	13	9	2	0	0	100
	1 000 acre-ft. per month.	0	0	23.3	46.6	81.6	77.8	66.1	50.5	35.0	7.8	0	0	388.7
	Acre-feet per month per second-foot.	0	0	17.1	34.3	60.0	57.1	48.5	37.1	25.7	5.7	0	0	285.5
Actual total..	1 000 acre ft. per month.	14.9	15.0	29.0	50.1	77.8	79.0	77.0	49.5	27.8	22.5	16.5	15.6	474.7
	Acre-feet per month per second-foot.	11.0	11.0	21.3	36.8	57.1	58.0	56.6	36.4	20.4	16.5	12.1	11.5	348.7
Yield useful..	1 000 acre-ft. per month.	0	0	23.3	46.6	77.8	77.8	66.1	49.5	27.8	7.8	0	0	376.7
	Acre-feet per month per second-foot.	0	0	17.1	34.3	57.1	57.1	48.5	36.4	20.4	5.7	0	0	276.6

The relative values of different water rights appraised were determined by superimposing on the hydrograph of the water right, as obtained by measurements over a period of years, the irrigation demand curve of the district, and their relative value was determined to be strictly in proportion to the number of acre-feet which each right would produce within the demand curve of the district.

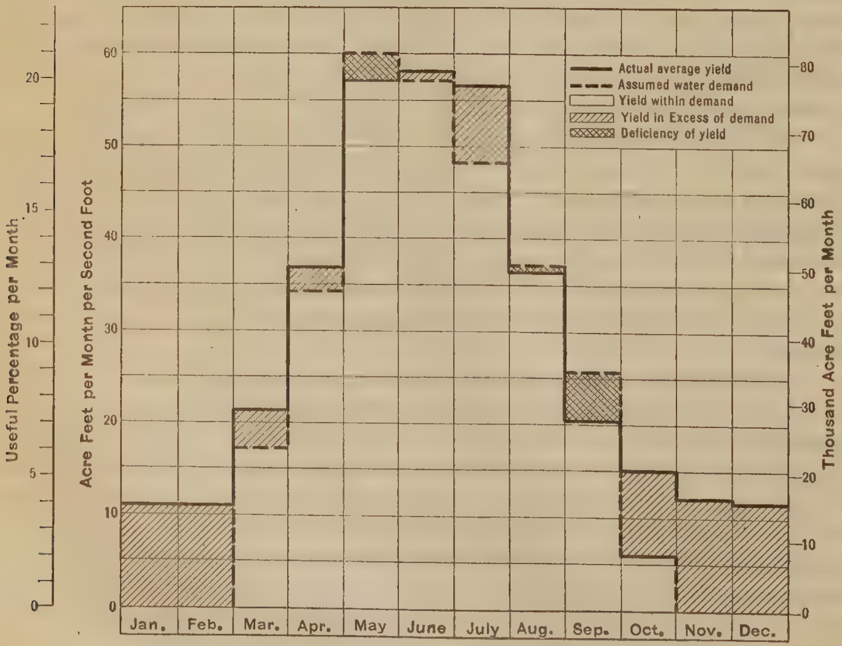

FIG. 2.—AVERAGE ACTUAL YIELD OF THE WATER RIGHT.

The final conclusion was that water rights from the San Joaquin River, for acquisition by the San Joaquin River Water Storage District, delivered at the head of the distribution canals, were worth $30.40 per acre to be irrigated with a normal demand of 2.45 acre-ft. per acre, or were worth $18.58 per acre-ft. of "safe net yield" obtainable during the minimum year of record.

CHARLES R. HEDKE,* M. AM. SOC. C. E. (by letter).—In the early stages of Western irrigation developments water was clearly provided to supply a local need in agricultural products for the margin between the cost of production and returns therefrom, at that time. It was simply a case of values then possible between demand and supply. The resultant of those two general forces constitutes the same basis now and will hold the forces making all future water values. Formerly, this basis of valuation was not affected by the many factors now involved but with the great growth and expansion of the country and its agricultural industry it can be readily seen that water values now are not such a simple matter. They have become quite complex. It is suspected that every hydraulic engineer has come sternly face to face with naked water values and has been disappointed in his search for information, so the courage of Mr. Field in opening the large subject should not go unmentioned.

Water is one of the natural resources of the country. It is indispensable for plant and animal life. Under the energy from the power station in the sun it forms the transportation system of all plants. It is continually moving, through evaporation and precipitation, and those phenomena constitute the fundamental water cycle in Nature where they are equal and opposite in direction. There is no permanent loss of water anywhere and Nature has provided its purpose and use. Like in all other natural resources, Man is undertaking to obtain from water its maximum use and value. In its extended use additional values are created, as certainly as for the soil or the minerals. As soon as the first additional value is made ownership will attach. Public ownership is natural and essential in the outset for water, just as it was for land, but there is no more reason why water should forever remain the property of the public than it is for the land or the minerals to be common property forever. No section of this country has escaped the inevitable result, yet Colorado is the only State which has shown the courage to clearly recognize it and openly declare water subject to private ownership. It is natural, therefore, that Mr. Field, a former State Engineer for Colorado, should complacently open the unsettled subject without ever mentioning ownership. A peculiar situation thus presents itself.

Within the general plan of Nature, it appears that two fundamental natural resources are necessary for the production of crops—climate and soil—and they have been properly related in Nature for the production of each particular crop. The understanding of the proper correlation of all the factors in this combination constitutes the science of agriculture. This is perhaps the oldest science, but it still appears that Man knows very little about it. Present knowledge places temperatures and rainfall as the most important factors in climate and holds that these fix the natural locations for the growth of each crop. Thus it is that wheat, corn, cotton, rice, cane, or apples, peaches, oranges, bananas, or strawberries, blackberries, coffee-berries, or the various nuts, vegetables, oils, drugs, and woods appear with a distinct zone for their natural and economical production.

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Under irrigation there is obtained the control of moisture for the growing of plants, and similar soils are found in each temperature zone, and yet crops are limited under irrigation; so it must be concluded that temperatures, which measure the heat energy available, are the controlling factor in Nature's crop zoning. Apparently some crops require a large amount of heat energy for their growth; others are suited for less heat; some want it between certain low limits, others between higher limits, and still others between extremely high limits and can stand no low limit at all. Such limitations appear to fix their production areas, provided the other factors of water and soil are suitable.

The writer believes that a definite and fixed general relation exists between the heat energy required for reproduction of a crop and the water requirements for that crop. It is further indicated in Nature that the same amount of heat energy requires the same amount of moisture supply in all plant production. Under that theory it would take the same amount of heat energy and water supply to produce a crop of oats in Texas, Colorado, or Alaska, all other factors except time being equal. The water requirements between crops then depend on their heat requirements. Large heat requirement crops will need large water requirements. High heat requirement crops may not necessarily need large water requirements. The essential factor seems to lie in the initial growing point for the plant as that appears to fix the point in Nature's energy supply which is required to start growth and place it in the zone of temperatures necessary for maturity and reproduction. All this is mentioned here to show that in the last analysis the value of water will lie in each particular kind of crop and the time required for its production to demand value. An acre-foot of water, in the Colorado River Basin, used in Central Park, Colorado, for the growing of native hay, field peas, or perhaps potatoes, all low and short heat requirement crops, will have a certain value, but if that acre-foot were used in the Imperial Valley, for the production of early lettuce, celery, potatoes, or citrus fruits, it would have quite another value, on account of the difference in time and kind of production.

The method proposed by Mr. Field for the evaluation of water rights does not go far enough. It is too local, because he bases it entirely on the $6 000 000 estimate given as the value of the Fort Lyon Canal property, but does not show how or why it should be that amount. It is the basic figure which is important and how that was determined, which is essential in the problem; also just how and when that value was created, would be of interest. Furthermore, in the computation used for volume delivered, an average quantity value is obtained, but is not made the final basis of value as the delivery did not cover the full year, and so a correction was made therefor. In that section it is believed water is not used for irrigation during the entire year, so there is no right attached for a full year's diversion and storage rights probably intervene to cover that unused period. It does not appear sound to extend values on that basis and the first value figure obtained by the author stands on a firmer basis than the later one.

It may be of interest to follow the economic steps necessary to create a real water value from its extended use by irrigation. The owners of land, in every semi-arid section of the West, with possibilities of obtaining a supplemental supply of water from a stream, in order to assure full and certain production, have thought, talked, and even taken what action they could, for some step to attach the possibility of a right to the use of some water to their land. A single owner of land soon discovered that he alone could rarely undertake such a task and hence with his neighbors, in mutual organization, the early developments were made. This form of development imposed heroic struggles and has since been supplanted by better means. The history of changes made in the developments by irrigation have been ably recorded and do not need to be repeated here. However, from the very beginning, the use of the water has been made the basis for a value and its contemplated use made a potential security aspect of considerable importance.

At the outset, for all irrigation properties, the valuation of the water can only be justified on the basis of the cost of the development. Unfortunately, the potential value of water was seen, grasped, and capitalized by speculators, who added it to the land values, and from that practice has grown the troubles from most land settlements under irrigation properties. The real value of water is created by its use and is represented by the actual production of crops the value of which is sufficient to show an attractive margin over the cost of production or over the returns from other competing areas growing similar crops. It does not come into existence until tried and proven use has been made and so should accrue to the party whose work and effort created that value. Any other basis is economically unsound and it appears the other viewpoints have led to much of the present irrigation trouble. Leave the possibility of a reasonable value of the water to accrue to the settler of the land under irrigation, who must make the value, and a changed aspect on most irrigation developments will be noted.

The kind of crops grown and the time of their possible production forecast the basis for the fundamental difference in the value of an acre-foot of water for irrigation purposes. Any crop grown, on the short-season areas, will come on the market after all other areas of longer season have matured similar crops. For staple crops that handicap may not be so serious. What is perhaps now more depreciating is that such short-season areas are sharply limited in the kind of crop grown so that the areas above them, in the scale of thermal location, contain less hazards and more advantages, which give the latter a better competitive position and thus reduce the value of water in the lower scale area. The oversight of such forces might be pointed to for some of the difficulties inherited by the present Reclamation Bureau. Its recognition then might furnish the basis for a defensible adjustment in their properties, between which relative water values could be determined and these values credited to the user by deduction from the construction cost, when apparently the water value was permitted to be attached. There is little possibility of finding a water value in some properties, with costs of construction and locations known to all irrigation engineers; and this does not mean all Government projects, either.

For some time, the writer has had a growing conviction that the most essential feature in the feasibility of an irrigation development lies in the condition that a clear water value can be reserved for the settler, as compensation and his margin, for the courage, self-denial, effort, and time it will take him to create the value of water for irrigation purposes. If a standard for such a margin could be agreed upon as just in all developments it would soon be quite clear in which zone of thermal location the irrigation developments in the near future would be undertaken.

In the same temperature locations, and under similar crop production, the several areas have a varying demand for water due to the difference of the natural water supply furnished by rainfall. The Imperial Valley of California and South Texas have a very close thermal position and their crops will be much the same; yet with the same consumptive use of water by cropping, which, for illustration, is taken as 36 in. applied to the land, the Imperial Valley obtains little if any direct water supply from rainfall, while South Texas will average (in the effective support) about 15 in., from its rainfall supply. In one case 3 acre-ft. of water are required to produce the crops, whereas in the other case, 1.75 acre-ft. will produce the same results.

From such practical conditions, which vary for every section, it would seem that a differing value of water exists for each section, and that it does not even remain stationary, but will rise and fall as the margins in the crops produced fluctuate.

FRED C. SCOBEY,* M. AM. SOC. C. E.—In his discussion of the value of reservoir rights the author does not make a distinction that appears essential. A great difference in the right to obtain water from a reservoir lies in the terms of the contract. Does the right entitle the holder to certain space in the reservoir or does it entitle him to definite quantities of water from the reservoir? In California and elsewhere, a number of contracts have been made in recent years whereby one organization agreed to pay to another a certain amount per acre-foot for water delivered—not space in the reservoir but tangible water.

On the other hand a favorite form of agreement entered into several years ago, particularly in Colorado, entitled the purchaser to a certain number of acre-feet of space in the reservoir. If full storage was obtained in any one year the presumption was that water to the extent of the right was started flowing from the reservoir to the lands of the purchaser, who stood the conveyance losses en route. If full storage was not obtained in any year the available water was pro-rated. The space in one well-known reservoir on the South Platte River was sold on the basis of a capacity of about 85 000 acre-ft. As constructed, and as further limited by action of the State Engineer, the reservoir has never had an available capacity of more than about 30 000 acre-ft. Here, the available capacity was only about one-third the assumed paper rights. In "short" years this is, of course, further reduced. Finally, what appeared to be a good and sufficient water right dwindled to merely a supplementary right.

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Another case is that of an organization in Utah that sold to another some space at the top of its reservoir. Moreover, it was a space that had seldom or never been filled. In cases like this the numerical volumes of prospective water may be such that a right looks good and sufficient to a land settler, and he does not find that space and water are not synonymous until he fails to receive the latter. When evaluated a distinction must be made.

There are, of course, many reservoirs that receive a full supply practically every year. For these, the consumer's rights need not be discounted. However, there are also reservoirs for which rights have been sold, that are worth only a small part of the same right if it could obtain the water numerically described therein.

Lyman E. Bishop,* M. Am. Soc. C. E.—Water rights in the Antero Reservoir near Denver, Colo., entitle the holder to a *pro rata* share of the quantity of water stored each year. The farmers under the Highline Canal own and control 11 254.6 acre-ft. of storage capacity rights in Antero Reservoir. These rights were sold to the farmers by the Antero and Lost Park Reservoir Company and were outstanding when the City of Denver purchased Antero Reservoir and the unsold water rights in it.

The Antero Reservoir water rights of the Highline Canal farmers were in litigation for several years. The Colorado Supreme Court in its decision, interpreting and deciding the value of these oustanding rights to the holders, stated in part:

> Farmers have "right to share upon a *pro rata* basis in the capacity of the reservoir in acre-feet as such capacity becomes available * * *.
>
> "* * * a right to use such proportional part of the water as might be stored with an agreement * * * to use reasonable diligence in filling * * *. * * * the contract nowhere provides that they are to receive an acre-foot of water per acre each year out of the reservoir.
>
> "* * * the rights of the farmers under their contracts can be no more than the right to use, on a *pro rata* basis, whatever water may be stored in the reservoir".

It should be noted that the Court states "water as might be stored" and "whatever water may be stored" and does not state or use the terms "is stored" or "is in storage" at any time or year. The City of Denver and the farmers are each entitled to their share and undisputed use, enjoyment, and ownership of their respective capacities. Each may use his stored water; each is entitled to his *pro rata* share of the water that "may be stored" at any time; each can accumulate and hold over his storage; and unless (or until) all the capacity that belongs to either is filled with water, there has certainly been no damage done to the other. Most assuredly, under Colorado irrigation laws, neither the city nor the farmers will be penalized for conserving water in storage any year and then be required the next year to *pro rata* or divide what has been saved with the other who has used his previous year's storage supply.

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The Antero Reservoir as constructed had a capacity of 58 601 acre-ft. At present (1928), on account of the weakened condition of the dam, concrete facing, etc., the State Engineer has limited storage to 33 276 acre-ft. Therefore, the "Highline farmers" who own water rights in this reservoir now control $\frac{11\ 254.6}{33\ 276.0} = 33.82\%$ of the total water stored in Antero Reservoir each year and the City of Denver owns $\frac{22\ 021.4}{33\ 276.0} = 66.18$ per cent.

The reservoir was completed in 1909 and water was first stored in May of that year. The City of Denver owns 47 346 acre-ft. and the farmers own 11 255 acre-ft. of Antero Reservoir's original constructed capacity. The reservoir has a decree of date, October 8, 1907, in the amount of 85 564 acre-ft. Of this decreed capacity, the farmers own 11 255 acre-ft. and the City of Denver owns 74 309 acre-ft. Irrespective of its usable capacity, the farmers under the Highline Canal holding Antero water rights own the full amount of 11 254.6 acre-ft. of storage capacity rights. If the reservoir were restored to its original constructed capacity of 58 601 acre-ft., the City of Denver would then own 47 346 acre-ft. of storage capacity rights instead of the 22 021 acre-ft. now controlled; and if the dam were raised to provide for an additional 6 ft. in depth of storage above the present constructed full supply level, the city would then own and control 74 309 acre-ft. of the then constructed and decreed capacity of 85 564 acre-ft. The date and the amount of the decree that has been entered for Antero Reservoir (October 8, 1907, and 85 564 acre-ft.) are of considerable importance. The storage decrees for Barr Lake enlargement, Prospect Reservoir, Horse Creek Reservoir, and Milton Lake are all of later date than the decree that has been entered for Antero Reservoir. This is quite significant because, since this decree is already entered and is a final one, the raising of Antero Dam to permit the storing of the decreed capacity of 85 564 acre-ft. would result in placing the demands for Antero storage ahead of the demands of Barr, Milton, etc. The South Platte River is the source of water supply for all these reservoirs.

Under the joint ownership and operation by the City of Denver of Antero Reservoir and Lake Cheesman the possible storage available for Antero Reservoir has been estimated in detail by the speaker for each month of the years 1900 to 1925, inclusive. This estimate of possible storage reveals the following as available annually for Antero Reservoir:

Mean (26 years)…………………	20 515 acre-ft.
Maximum (1914)………………	58 601 acre-ft.
Minimum (1908)………………	3 845 acre-ft.

The records of Antero actual annual storage since 1909, show the following:

Mean (19 years)…………………	13 214 acre-ft.
Maximum (1918)………………	27 801 acre-ft.
Minimum (1911)………………	3 842 acre-ft.

From the foregoing brief statements it will be realized that the value and probable yield of Antero water rights are subject to several criteria. The usable capacity of the reservoir; the variation of the available gross storage; and the requirements and use of the stored water by the water-right owners, are the three principal arguments. All these considerations are subject to change and all changing results in variable yields for Antero water rights and correspondingly different values.

R. L. PARSHALL,* ASSOC. M. AM. SOC. C. E.—If there is some basis on which it could be said that a second-foot of water has a particular, definite, fixed value and that there is some relation existing between the value of a direct flowage right and storage right, it may be of interest to know that approximately 3 000 acre-ft. of stored water in the Arkansas Valley, in Colorado, was recently purchased at $2 per acre-ft. In the northern part of the State, near Fort Collins, the value of stored water varies according to supply and demand. As a minimum, water in storage sells from $40 to $60 per 1 000 000 cu. ft., while, at other times, the price is as much as $200. In June, 1927, stored water was sold for about $95 per 1 000 000 cu. ft., which is, roughly, $4 per acre-ft.

S. H. STIVERS,† M. AM. SOC. C. E. (by letter).—For some time past, the valuation of water rights has been the subject of extensive investigation and study, as well as of judicial review. This subject is of increasing importance as shown by the widely divergent views and decisions, in recent rate cases, and the present tendency of legislative bodies in the enactment of statutes affecting rates, earnings, consolidation, and mergers.

The writer's understanding of Mr. Field's method of valuation is that he first determines the total market value of the outstanding stock and other obligations. From the result so determined, he deducts the appraised value of the physical property and plant, the residue plus certain indebtedness being assigned as the value of the water right. Without raising any question as to the reasonableness of that value in this particular case, it is apparent that the method of determination is contrary to the generally accepted methods of valuation. It is, therefore, fundamentally unsound inasmuch as its validity is at least problematic in view of the decisions of the Courts in recent cases, and relates itself to the fundamental principles of valuation thus far announced by the Courts.

At one time valuation was defined as a determination, by competent public authority in the exercise of a governmental function, of a sum representing the total property invested in an enterprise. This sum, however, would not necessarily represent or indicate the relationship between the sum thus found and the property itself.

Rate-making is the most fundamental purpose of valuation, because it determines the earnings of the enterprise; and these earnings, when determined and fixed, will establish to a large extent the commercial value of that enterprise. The consideration of the basis of a valuation for rate-making purposes involves the fundamental principles of economics, political science, and law. The method must be sound from the economic point of view, or it will result in hardships and confusion. It must be correct from the point of view of political science and National policy, or great injustice may result to the utilities, to its patrons, and to that part of the public that has invested in its securities. It must be judicially sound, or constitutional legal rights may be usurped and the whole proceeding may be invalidated, because the power of judicial review over such valuation has become firmly established in American constitutional law. A valuation based on political expediency, however, is to be deplored from a scientific point of view and will inevitably result in a change in the National policy in the event that such a valuation receives the approval of the highest authority.

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It is apparent that theory as to correct "value" ranges from that of the economist, which is market value or exchange value, to the theory involving original cost of the enterprise and to the so-called prudent investment. It is submitted, however, that the judicial concept, in so far as it has been developed, is not strictly "value", and that judicial dicta, if not judicial decisions, might be construed to support almost any one of the theories that might be adopted. It will facilitate comparison of these theories to analyze the fundamental economic principles to which they apply and to which they give varying considerations and weight.

Market Value.—This theory runs through valuation literature from the discussions of the Georgia Commission, in 1880, to the present time (1928), and purports to be the economist's true concept of value. By no process of reasoning, can value in exchange be found in advance; it can only occur in the event of a sale, and if there is no sale, there is no value in exchange. An attempt to determine this value in exchange is merely an effort to estimate what such a value would or might be if a sale takes place. In the case of large utilities the so-called "market value" is rarely ever determined, because such properties are rarely subject to purchase and sale.

The market value of stocks and bonds does not necessarily indicate, with any degree of accuracy, what the sale value of the plant would be. The stock and bond quotations on the market are generally the result of offers to sell and bids to buy such securities as are offered for sale, and this floating supply usually represents only a small proportion of the total outstanding. A steady and consistent demand and a diminishing supply of the securities might indicate negotiations for control of the enterprise, and such control might be desired, or required, for reasons entirely foreign to the value of the property. On the other hand, if a large part of the outstanding issues was thrown on the market, the inability of the market to absorb them might result in a substantial decrease in the prices quoted or bid; and if bids were made for substantially large blocks of stock without a corresponding release on the part of present holders, quotations would be affected by purchases for speculative purposes, without regard to the value of the property represented by the securities. About the most favorable claim that can be made for this theory of value is that if the market is in exact equilibrium (an almost impossible hypothesis) the quotations represent the judgment of investors, or speculators, as to what the earnings and the probable speculative value of the securities might be. The value of shares in mutual irrigation companies may represent, in some instances, the fair value of the property because this value sometimes represents the amount the farmer is willing to pay for a certain quantity of water delivered at the point of use.

The correlative theory sometimes suggested for rate-making purposes is that capitalized earnings represent value. These earnings are a sum representing the value of the utility to its present owners, and, while this factor may have a decided bearing on exchange value, it is not exchange value itself. This rule must inevitably disrupt the question of reasonable profits, since it involves the so-called vicious circle that the reasonableness of rates depends on the value of the plant operating under reasonable rates.

Original Cost.—Many theorists hold to the actual investment theory of value. If it is assumed that money was invested, they claim that the actual money outlay is the basis of value; but investment is not necessarily limited to this narrow meaning. It can be interpreted as investment in property or investment in purchasing power. This is necessarily true and derives its significance from the general changes in price level. If such changes had not occurred, there would have been little or no necessity for any theory of valuation, because investment in terms of money would have satisfied both the legal and the economic requirements. The primary objection to considering investment in terms of money alone, is that it excludes artificially the rate base of utilities, railroads, etc., that are subject to statutory regulation, from the operation of economic laws, which are controlling in the case of all other classes of business. If rates are based on investment in terms of money and the price level changes upward, regulatory rates would become out of line with other prices, and the public would enjoy a purely artificial advantage, due to the accidental circumstance that the plant was built at a time of low costs. Patrons then will continue to pay rates that are unreasonably low as compared with prices in other classes of business in which they themselves are engaged. It has been repeatedly held by high judicial authorities that utilities subject to statutory rate regulation are entitled to share in the general increase of property values, and this rule seems to be less arbitrary and more economically sound than to adhere slavishly to the rule that investment in terms of money is the proper basis.

Such a rule is equitable, because, if the price level declines (even if the dollars received from the enterprise will be less than if rates were based on the investment in terms of money), this will be compensated by an increase in the purchasing power of those dollars in terms of other commodities. Money is used as a standard of value purely for convenience, and as a standard of deferred payments it is generally recognized by economists to be faulty. However, it is probably better than any other standard that might be adopted. Assuming that utility investment should be stable, it is no more certain that this stability should be in terms of dollars than in terms of purchasing power.

While investment in terms of purchasing power would probably be theoretically more correct than investment in terms of property, it would be difficult to ascertain and apply. The latter method will give nearly the same results as the former and it offers an easier means of determining cost of reproduction at present unit prices. Approaching the cost of reproduction basis from the investment angle, through the idea of investment in terms of property, its merit, namely, that it recognizes change in price levels, is retained, while the arbitrary and speculative features of the method are eliminated.

No hard and fast rule, or formula, can be laid down to govern the valuation of water rights, or of any plant or industry. The appraiser must determine all relevant facts that have a bearing on the value and give due consideration to all of them. This rule was first handed down by the U. S. Supreme Court in the case of Smyth *v.* Ames. This now famous decision enumerates the items of evidence to be considered in finding the value, but carefully avoids laying down any formula for applying them to the results sought. The Court simply states that all these factors must be considered, and then there must be a reasonable judgment based on a consideration of all relevant facts. The Minnesota Rate Case quotes this dictum with approval.

Water Rights Must Be Valued.—The question often arises, "Should water rights be valued"? It is clearly settled by numerous decisions to the effect that when water rights have been reduced to private ownership, either by appropriation, or purchase, such rights must be valued, as in the case of any other privately owned property. (See San Joaquin Co. *v.* Stanislaus County, 233 U. S., 454; Spring Valley Water Co. *v.* San Francisco, 192 Fed., 137.) A water right is private property and in a rate case the owner is entitled to have the value of such right included in the rate base. However, a distinction should be made between water rights that have been reduced to private ownership and contract rights for water. The U. S. Supreme Court, in the Indianapolis water case, includes the value of water rights in the total valuation.

Quantity of Water to Be Valued.—This will vary in different States, because of the statutory provisions of the State laws. Some provide that, to reduce appropriated water rights to private ownership, the water must be put to beneficial use; only so much as is put to such use, becomes the property of the appropriator, provided, of course, that this beneficial use is not limited by, or in conflict with, prior appropriations from the same source of supply.

A Part of Land Value.—If the water rights are incidental to the ownership of the land, or appurtenant thereto, they should be valued as a part of the land itself. (See Portland R. E. & P. Co. *v.* P. U. R., 1916 D, 876, 1022.) Such rights may be classified as follows:

(1) Those in which the source of water is on the land, and is controlled by the ownership in the land, such as percolating waters, springs from which water does not flow, or storm waters impounded in reservoirs.

(2) Those in which the source of water is on other land, but the right to use it has been attached to owned land.

It will be observed that, in the first case, the water right inheres in the land as a matter of law, and in the second case, as an easement on a servient tenement. If the land were sold, the water right would pass with the land unless reserved in the grant, as in the case of reservoir land and storm water. It is the writer's opinion that water rights of this nature should be valued as a part of the land. This condition may vary in the different States, due to State laws.

The question does not arise with respect to rights of appropriation, because ownership of land is not necessary in order to acquire and retain a right of appropriation. Such rights have a value separate and distinct from the value of any land.

Commissioner Loveland, in his opinion (Santa Cruz, P. U. P. 1915, F. 768, page 792), states:

> "* * * This is a case of the appropriation of water from the banks of a natural stream by an appropriator, who did not own land in connection with such appropriation. The evidence in this proceeding shows: First, there is a live stream; second, the doctrine of riparian rights and appropriations can be applied in this case; third, the water company is not the owner of land in connection with such diversion of water. I am of the opinion that the water rights have a value separate and distinct from the lands.
>
> "* * * after considering these different elements of value, I recommend that some such sum be allowed as these water rights must have cost if purchased to-day."

Value at the Source.—The writer is of the opinion that a water right should be valued at the source of supply. The water itself may be diverted from the original source and may be led for long distances through ditches or pipe lines and put to beneficial use at some other point. Nevertheless, it appears equitable that the proper value is the value at the source and not that at the point of use. This is well sustained by the Courts for (in Spring Valley Water Company *v.* San Francisco, 252 Fed., 979, 984 (1918)), the Court has stated:

> "* * * Under such circumstances to allow the company the full value of water-shed and storage reservoirs and add thereto the value of the water when collected and stored is to place a double burden on the rate payers. The plaintiff is undoubtedly entitled to the full value of its storage facilities, but, in addition thereto, it is only entitled to the fair value of the water rights in their natural state * * *."

Elements of Value to Be Considered.—Cost.—Some of the State commissions seem to be unduly influenced by the theory that, originally, the States permitted rights of appropriation to be freely acquired. They conclude that it is not in the public interest to permit these water rights to be valued, in rate-making cases, at a sum in excess of their cost, regardless of the fact that the right in question may have a value entirely different in exchange or in condemnation proceedings. It is presumed that this is predicated on the theory held by some commissions, that only actual cost to the utility can be assigned to franchise rights. The writer is unable, however, to identify a water right reduced to private ownership, with a franchise right, even though such ownership rests in a public utility under governmental regulations.

In its first decision involving the value of a water right, or water power right, the California Commission held that if the appropriation was made for a public use (apparently public utility use), it was held in trust for the public, and that the gift or donation by the public could not be capitalized against it. On the other hand, if the appropriation was made for private use and later acquired by the public utility, it should be valued at its cost to the utility. (See Northern California Power Co., 1 Cal., R. C. D. 315.) After the decision of the Supreme Court in the San Joaquin case, that body frequently took occasion to deplore that holding (see San José *v.* San José Water Co., 4 Cal., R. C. D. 1101; *re* City of San Diego, 4 Cal., R. C. D. 902; *re* City of Glendale, 4 Cal., R. C. D. 1011, 1019), as in the Glendale case, it is stated that a commissioner should make such a finding as he believes to be fair, "always limited by what he thinks higher authority will permit, which higher authority in many instances, we are led to believe, does not give the conscientious thought to this subject which it should." In some of its subsequent cases, while the written opinion was orthodox, it is impossible to determine what sum was included in the valuation of the whole property for the water rights. Appraisers cannot, however, escape the decision of the Supreme Court (San Joaquin case) that the cost of the right is immaterial, and that the Supreme Court of Idaho held (see Murray *v.* P. U. C.) that the cost may be considered, but is not conclusive as to value.

It may even be admitted that if the water right were valued at, or near, the date of its acquisition, the cost might be an indication of its value. If the cost of the right was small, its value was probably small, even though its utility and potential future value was high. Subsequent thereto, changes have usually occurred to such an extent as to alter completely conditions surrounding the right. Furthermore, there is generally a quasi-monopoly feature in water rights as well as in land, since the supply is usually limited and the demand in a developing community is increasing. The utility of the right for particular purposes may have greatly increased; its irrigation utility may have become higher, due to improved market conditions for crops, changes in the art of cultivation, and the development of new crops, thereby making it more profitable to irrigate, and increasing the amount the farmer can afford to pay for water. Additional or different utilities for the use of the water may have developed. All these changes enter into the new value of the water right and preclude the possibility of using the cost as value. Moreover, the owner (whether an individual or a public utility) is entitled to the increased increment of value. It can scarcely be said that there is any more reason for confiscating this increment in the value of water rights which have accrued in private ownership, than for confiscating the increment in land value.

Next Available Source.—A method advanced by some authorities is the use of the capitalized savings over the cost of bringing water from the next available source. This can only be based on the assumption that the right is worth what it would cost to bring water from the next available source and convert it to the same use to which the water right under consideration is devoted. This assumption seems contrary to fact, and its effect might be to pyramid the strategic value of the right in use, a result which is indefensible under the decisions heretofore discussed (Spring Valley Water Co., *supra*).

Even conceding that the water must be brought from the next available source, this method is purely relative and gives little or no consideration to the utility of the right being valued, being concerned only with its relative utility in terms of the utility of another right. The value of the second right can only be measured, by this method, in terms of the value of a third right, and so on, *ad infinitum*. If this theory has any merit at all, it can be extended to produce a value, determined by the savings over the most expensive source, rather than over the least expensive. This method appears in conflict with a Court decision on this subject.

In the case of Brunswick *v.* Maine Water Co. (99 Me., 371), it was sought to condemn the property of the Water Company, including its water rights, and the Court said:

> "* * * A water company undertaking to supply a community with water is bound to do so wisely and economically. It is bound to take advantage of practically natural facilities. If there is more than one source of supply, other things being equal, the community is entitled to have the least expensive one used. * * * For instance, if water can profitably be served from a nearer source of supply, the Company ought not to be permitted to charge a higher rate based upon the expense of bringing it from a farther and more expensive source."

Furthermore, this method, being purely relative, can be reversed and the water right, not being appraised, can conceivably be given a negative value when compared with zero for the right being valued. Having adopted this comparative method, the question is, which of the two sources shall be taken as the standard, the one which the utility would use in the exercise of ordinary business judgment, or the one which it might use if it were imprudent enough to do otherwise. In the absence of an elaborate survey of all visible and underground resources there is no definite assurance that the assumed supply is, in fact, the next available source; and if all this assumption were definitely assured, the theory is still invalidated by the necessity of valuing the right at its source and in its natural state.

Substitute Plant Theory.—A somewhat similar basis sometimes adopted is the saving in the cost of operating by water as compared with coal or oil in valuing water-power rights. This covers more ground than the "next available source" method, in that it resorts to a substitutional means of manufacturing electrical energy. Some decisions can be found that favor this as a measure of value, while others disapprove it for the following reasons: It is comparative only; it is unstable in that its value is reflected in the changing prices of the fuel; and it deprives the public of all benefit from a natural resource to which they are adjacent. Some commissions hold that this saving should be shared with the consuming public, while one of the principal arguments against it is found in the decisions that it results in the maximum possible value and that it ignores the all-important element of supply and demand.

Market Value the Test.—Practical economics, as well as a long line of Court decisions, appear to demand that the market value of a water right is the true test. That is, it is the true market value determined from a consideration of all relevant facts in view of all their available uses and all the circumstances affecting their value.

It is important to distinguish between the market value of the right and its utility, or usefulness, to the owner or user. Market value may be denominated as value in exchange and can be said to be determined by the varying utilities to possible purchasers and affected by the doctrine of supply and demand. A common example of the distinction between value and utility, is the air. Its utility is inestimable, since no life can exist without it, while its value is nothing, since there is more than enough air for all who require it. Another example is water; in most Eastern States, where the supply is greatly in excess of the demand, it is of practically no value, but its utility is great.

A water right has a utility to sustain life, another utility for domestic use, another for railroad purposes, another for irrigation purposes, another for developing electrical energy, and another for stock raising. These utilities vary considerably in value, and represent what it is worth to users of water for the purposes indicated. The utility seems to be a logical starting point in any computation of value which may be said to be a ratio between supply and demand, operating on the highest utility represented by the market. It is recognized, of course, that the adaptability of a water right for various uses is a necessary element in the determination of market value, but its adaptability for any one particular purpose, or its usefulness for that purpose, does not necessarily govern its market value. In condemnation cases, it is a clearly established rule that the object of the inquiry is the market value of the property taken, in view of all its available uses, and not its usefulness to its present owner, to the taker, or to any other person.

The market value of water rights will vary widely in different sections of the country, because in sections of normal rainfall such rights have little or no commercial value for ordinary purposes, while in arid or desert sections almost any water right has a very decided value. The writer had occasion to undertake the valuation of a water right in the arid or semi-arid sections of New Mexico, and gives herewith the salient factors used in the determination of its value. This is not put forward in any sense as a criterion of a universal method, but is submitted as one method of valuing a water right of this character.

Description.—This water right is owned by the El Paso and Southwestern Railroad Company, which extends from Santa Rosa, N. Mex., in a general southwesterly direction, through El Paso, Tex. The line is located along the eastern slope of a spur of the Rocky Mountains and traverses both a high rolling and extensive flat arid area, or mesa.

This vast desert area, almost barren of vegetation, except for the usual desert mesquite growth, is practically without natural drainage, with the exception of small streams which originate on the slopes of the surrounding mountain ranges during the short periods of rainfall and snow melting. From Santa Rosa westward, the railroad is constructed on the desert uplift, until the Corona Divide is reached (about Elevation 6 725). It lies within what is known as the Tularosa Basin, which extends from this point to the southern limit of the mesa east of El Paso.

From a study of the map of New Mexico, it will be seen that the eastern part of the State is drained by the Pecos River, with its head-waters in small tributaries originating in the mountain ranges of San Miguel County. This stream flows south, crossing the railroad near Santa Rosa and passing well to the east of the Sacramento and Sierra Blanca Mountain Ranges. The central part of the State is drained by the Rio Grande, with its head-waters in the mountain ranges of Taos County. It flows south into Texas, crossing the railroad at El Paso, and passes west of the Oscura, San Andres, and Organ Mountain Ranges.

The Tularosa Basin lies between these two drainage areas. It is flanked on the west by the Oscura, San Andres, and Organ Mountain Ranges, and on the east by the Jarilla, Sierra Blanca, and Sacramento Mountain Ranges.

According to a rational conception of geological reports, the present Tularosa Basin was, in ages past, the peak of a great uplift which divided the drainage basins of the Pecos River and the Rio Grande. Subsequent settlement created a low valley with rough and rocky mountain ranges on either side and from the deposits of ages, this basin was formed as a gently rolling mesa. The rock strata of the mountain ranges on the western side of the basin slope toward the west and those of the mountain ranges on the eastern side slope toward the east. Thus, the seepage water from the western range finds its way into the Rio Grande drainage area and that from the eastern range flows into that of the Pecos River, leaving a gulf of barrenness approximately 60 miles wide and about 150 miles long, which is without a drainage outlet of any kind.

The foothills of the surrounding mountains and the floor of the Basin itself, are underlaid with strata of limestone and gypsum deposits, while the surface is highly impregnated with salts and alkali. Extending 10 or 15 miles west from Coyote, N. Mex., in Lincoln County, there is plainly visible an ancient lava flow from a volcano now extinct. This lava flow continues south parallel to the railroad for approximately 45 miles. At this point, a small stream emerges from beneath the lava bed. The water, however, is so impregnated with salts, gypsum, and alkali as to be unfit to water stock, for domestic use, or for use in steam boilers. Immediately south of the lava bed are extensive beds of powdered gypsum, which offer a striking contrast to the black lava flow when viewed from a distance. The Tularosa Basin has the average rainfall of a general desert country. It is said that this area constitutes the largest inland basin on the North American Continent with absolutely no run-off.

Bonito Water Right.—This right is one of appropriation from the waters of Bonito and Eagle Creeks. These creeks constitute a part of the Hondo River drainage area, which flows into the Pecos River near Roswell, N. Mex. The water has its source in the upper reaches of White Mountain Peak (Elevation 12 000) which is a part of the Sierra Blanca Range. The lower part of this mountain contains vast deposits of gypsum and limestone strata, both of which are soluble in water and, when dissolved, produce a hard water that has a tendency to form hard scales when used in steam boilers. The upper part of the mountain is composed almost exclusively of a hard granite formation which contains little or no soluble matter.

Prior to the diversion of this water for boiler use, the railroad had taken water from the Pecos River and from drilled wells. The water thus obtained was unfit for boiler use, and efforts to treat it chemically failed to the extent that the treating plant and the Pecos River supply were abandoned. The water from Bonito and Eagle Creeks is admirably suited for boiler use since it comes from the melted snow on the mountain and contains only a slight chemical content. This water is conveyed by a gravity pipe line, from intakes on Bonito and Eagle Creeks to the Nogal storage reservoir. From the reservoir, it flows by gravity to the railroad at Carrizozo and Coyote, N. Mex., a total distance of approximately 46 miles. From this point the water is pumped north along the railroad to Vaughn, a distance of approximately 100 miles. The Nogal storage reservoir has a capacity of approximately 400 000 000 gal.

At the time the carrier decided to develop this water for railroad purposes, a part of it had been appropriated by farmers, who were putting it to beneficial use by irrigating the small farms in the valleys along the creeks. The quantity of water thus used is unknown, because no regular system of measurement had been practiced. In 1907, the carrier purchased all the irrigated lands on Eagle and Bonito Creeks as far east as the Fort Staunton Military Reservation, and abandoned the use of water for irrigation purposes in order to divert the water into the pipe line.

The carrier, by purchase and appropriation, completed its title to 5 sec-ft. of water from Bonito Creek and obtained authority from the State to change the point of diversion and to convert the water from irrigation to railroad use. It also perfected its title to 6 sec-ft. of water from Eagle Creek with the right to use it for railroad purposes. The only adverse user of water from these creeks is the Military Reservation, which is entitled to 89 000 000 gal. annually. Under an agreement with the Federal Government, the carrier constructed a branch pipe line about 2 miles long, to and for the use of the Military Reservation.

At the time this appraisal was made, the annual flow of Bonito Creek was more than the quantity of water to which the carrier is entitled, while the annual flow of Eagle Creek is insufficient to enable the carrier to obtain its legal share of water. Thus, the water to which the carrier is entitled is 11 sec-ft., or 2 596 000 000 gal., while the water actually available to the carrier is:

Bonito Creek........	5.000 sec-ft., or	1 180 000 000 gal.
Eagle Creek.........	3.346 " " , or	799 000 000 "
Total	8.346 sec-ft., or	1 979 000 000 gal.

This is equivalent to 5 395 000 gal. daily.

At the time of this appraisal the full quantity of water to which the carrier is entitled, was not actually diverted from the streams into the pipe line, because that quantity was then in excess of the actual requirements of the railroad. The road, however, is holding this water for future needs, and to provide for storage for use in the dry seasons.

To accomplish this purpose, water is stored in Nogal Reservoir during the period when it is plentiful. The water in excess of the capacity of the pipe lines is permitted to escape through the natural channel of the stream and is used by the carrier's agent for irrigation purposes on the Bonito Farms, which are owned by the carrier. This precludes the possibility of adverse appropriations, through beneficial use by others farther down stream. The writer is informed that these farms are operated at a loss to the company. It was the appaiser's opinion, however, that inasmuch as the carrier should prepare for future expansion of its business, with resulting increased requirements for water consumption, all this water should be considered as "used and useful" for transportation purposes.

Value for Alternate Use of Irrigation.—Assuming the availability of sufficient land to consume, for irrigation purposes, the water owned by the carrier, the measure of value of the water right would be the difference between the value of the land with and without water. From a minute study of other irrigation projects in this locality, giving consideration to the similarity in average annual rainfall, it was found that 2 acre-ft. per annum would be the correct measure of the quantity of water necessary to irrigate 1 acre of land. Since 8.346 sec-ft. is equivalent to 6 043 acre-ft., the water right would irrigate 3 021 acres of land. The difference in value for the land was appraised at $275 per acre. This produces a value for the water right of $830 775. The writer concluded that this figure fairly represented the value of the water right for irrigation purposes and would fairly represent the cost to the carrier, if title to all the water was vested in private parties, and should the carrier attempt to purchase the right in its natural condition and develop it for railroad purposes, as of the date of appraisal. This figure does not give consideration to the so-called excess cost of acquisition almost invariably incurred by a railroad in the purchase or condemnation of real property.

Some authorities believe that, inasmuch as the water is used for irrigation purposes for only about one-half the year, the value reflected by this method should be doubled because the carrier makes use of the water for the entire year. It will be observed, however, that the acreage which the water would irrigate, is based on a calculation of the total quantity of water that the carrier would obtain from the flow of the streams, for a full year, under its right of appropriation. In order to utilize the water that runs during the non-irrigating season it would be necessary to collect and store it for use during the growing season. The value of the right would then decrease in direct proportion to the increase of the cost of storage and distribution of the water.

Original Cost.—The cost of land, water rights, and pipe line easements (inseparable) to the carrier was approximately $245 000. The quantity of water to which this purchase entitles the carrier is unknown because its title to the water was afterward perfected through appropriation. This purchase was made sixteen years prior to the date of valuation.

The writer made a close investigation of all water supplies in the Tularosa Basin, and as a result of this study reached the conclusion that no other source of supply was available that could have been secured at a reasonable money outlay. Three mountain streams, from which water is obtained, were found to exist: La Luz, Fresnal, and Tularosa Creeks.

La Luz Creek and Fresnal Creek originate in the Sacramento Mountains at La Luz, and Alamogordo, N. Mex., and serve the two towns as well as the immediately adjacent farming sections. All this water was appropriated many years ago and it would be impossible to obtain title to it except through purchase of all the land entitled to water, together with the improvements thereon. The railroad obtains some water at Alamogordo for $500 per month. It is necessary to treat this water before using it in locomotive boilers.

A reservoir on the Fall Ranch, located a few miles east of Three Rivers, N. Mex., impounds the waters from small streams originating in the Sacramento Mountains. The carrier at one time purchased water from this source on an annual basis; subsequently, it purchased a permanent water right.

The carrier owns the water supply at Orgrande, N. Mex., which is taken from the Sacramento River and is conveyed by pipe line to a reservoir on the Jarilla Mountain, from which it flows through a gravity pipe line to the railroad track.

A small stream of water originating in mountain springs, is brought into the Town of Tularosa by open ditch and is owned by the town and its inhabitants.

At other points, deep wells had been drilled to secure water. These ranged from 850 to 1 400 ft. in depth. The water obtained at a depth of less than 1 000 ft. was generally gypseous and contained approximately 3 000 parts of incrusting salts per million. Water below this level, while not extremely hard, had a concentration of soluble salts about seven times that of sea water, and contained calcium and magnesium in large quantities. The well water was generally unfit for boiler use.

Investigation developed the fact that water was sold from the Tularosa supply for irrigation at approximately $1 650 per water right. A "water right" (which was sold as a part of the land) was found to be that unknown quantity of water required to irrigate 10 acres. While the Tularosa right is somewhat removed from the territory of the Bonito water right, and the two problems are not identical, it is described herein as general information regarding land and water rights in this section.

Sale of Water Right from the Fall Right to the Carrier.—This was a sale of a daily flow of 75 000 gal. of water for $45 000, or at the rate of $600 per 1 000 gal. daily. The sale price includes a 3-in. pipe line about 2 miles long. The agreement provides that the grantor shall have the joint use of the pipe line with the carrier, but must deliver to the carrier 75 000 gal. daily. The water is delivered at the track, or point of use, and not at its source. The writer applied this sale to the Bonito water right as follows: Available supply, 5 421 000 gal. daily; capacity of pipe line from intakes to Nogal Reservoir, 3 500 000 gal. daily; and capacity of pipe line from reservoir to the railroad, 1 500 000 gal. daily. Delivered at the point of use at the rate developed by the sale, 1 500 000 gal. daily results in a value of $900 000. From one point of view, the Bonito right is more valuable than the Fall right, due to the fact that it insures a supply of water during periods of protracted drought, and provides for the carrier's requirements upon the expansion of business. On the other hand, the Fall right may be considered as more valuable, for a similar quantity of water, because it is only necessary to pipe this water about 2 miles, while the Bonito right requires a pipe line of 46 miles to the first point of use. The writer is of the opinion that this sale does not represent fairly the value of the Bonito water right, in its natural state. That the value derived by this analysis of the Fall sale is fairly close to that developed for irrigation purposes, is merely a coincidence. Were the carrier's present requirements reduced to one-half and that quantity delivered at the railroad, the value deduced by the application of this sale price would be reduced by one-half.

The carrier received in 1918 an offer from the "Ruidoso Water Users' Association". The Ruidoso River is a tributary of the Hondo and is a few miles south of Eagle Creek. This offer was for 4 sec-ft., valued at $300 per acre, on a 3 000-acre basis, or $900 000. The writer believes that the offer was based on an erroneous assumption. Four second-feet would irrigate about 1 500 acres of land and not 3 000 acres. On this basis, the carrier's water right would be valued at $1 877 850.

Considering the cost of operation on the railroad prior to the development of the Bonito water right and comparing it to the cost after the water from this source was put into use, and then capitalizing the difference, the result would be a value of approximately $3 000 000, if it were assumed that all the savings in operating expenses were due to the use of this water. It seems quite probable that the greater proportion of the decreased cost of operation can be directly attributed to the use of the water.

Consideration was given to a number of sales of water rights used for watering stock. These sales averaged $695 per 1 000 gal. daily, based on the purchaser's requirements of 7 000 gal. per day, which is the quantity required for 560 head of cattle, the maximum number that could be watered from one spring. In purchases of this character, the rancher satisfies himself that he will get a range supply of 7 000 gal. daily; he is not interested in any greater quantity. He would pay the same price for an assured supply of 7 000 gal. daily as he would for a supply several times that amount.

After giving consideration to all the conditions surrounding this water right, its available uses, original cost, future requirements, etc., the writer appraised it at $913 000, at its source and in its natural state. This was a valuation for rate-making purposes.

C. E. Grunsky,* Past-President, Am. Soc. C. E. (by letter).—The author has attempted to find a simple, generally applicable method of ascertaining the value of a water right. The attempt is laudable, but the effort will be in vain. One main difficulty with the particular method suggested lies in the fact that no rule can be found for apportioning the aggregate value of all intangible elements to the individual elements. Moreover, under the American system of controlled rates, the intangible elements of public utilities acquire value only when rates are fixed sufficiently high. If the earnings will not yield income in excess of operating expenses plus the ordinary cost of money, there will be no intangible values.

A water right, in many respects, is similar to a franchise. It is a grant by the public. It may have indefinite or only a limited life. Its acquisition may have cost something or nothing. Under certain circumstances the water right may acquire a recognized market value, as will be the case if all the water that is available is needed for irrigation.

Frequently in acquiring a water right, it becomes necessary to quiet adverse rights. It may be necessary, for example, to compensate riparian owners for the actual or the prospective damage to their property which would result from an up-stream diversion. All legitimate expenditures made in acquiring a water right (with due allowance for any change which may have taken place in the purchasing power of the dollar) should appear in the rate base.

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The term, "rate base", as herein used, must not be confounded with market value. The aggregate market value of a public utility property results from its earnings and its earnings, in turn, depend on rates. The rates should have some fairly definite relation to the rate base. The value resulting from the rates cannot be used.

Both the Courts and the rate-fixing authorities have definitely assumed the attitude that, when water rights are involved, earnings should be allowed with due consideration of the fact that the water right may have value; but this principle is not thus expressed in the decisions. As the author states, no method has yet been found for evaluating water rights which would be acceptable to the Courts, public service commissions, or valuation experts.

Sometimes recourse is had to a comparison with the cost of developing water in the same general region in which a water right is to be valued. The results are rarely satisfactory, because, in determining such average value, some costs which are above the average and others that are below the average, will be taken into account. It would be evidently absurd to give those whose development has cost in excess of the average a negative value. In places where water is scarce and water rights have been made a matter of bargain and sale, a definite basis can sometimes be found for estimating value. In such cases, the earnings that result from the use of the water become the measure of its value, and this value, in consequence, may assume a wide range. A determination of the value of water rights in such sections of the country will have an effect on the value of rights elsewhere, the question naturally being asked, "Why, if water commands a high price in one section of the country, should it not also command a similar price under comparable conditions elsewhere?" It seems obvious that whoever makes a successful development of water and puts it to beneficial use has done something worth while for society and is entitled to a fair reward. It would not be unreasonable, therefore, to make some allowance in the earnings, preferably based on the ordinary regional costs of developing water which, when capitalized, would be the value of the water right.

If this principle were accepted by the rate-fixing authorities it would result in removing indefiniteness relating to the value of water at its source. The value of a water right, if thus allowed and intentionally recognized and established by the rate-fixing authorities, would not be subject to such uncertainty, nor to the wide range so notable at present. It sometimes happens that, when all the water available is applied to beneficial use, the cost of development of some of the supply is far below that of the remainder. In such cases the value of the water right—if the cost of development is low—may be very large in comparison with the value (at its source) of the other water used in the same district. In such cases, strategic value is recognized, and this can be determined more or less satisfactorily by the comparison-of-cost method.

Another factor which deserves consideration when a water right is to be evaluated is the time during which the right may be exercised, and still another factor is the length of time that it may take to put the water right to full use. Attention is called to these matters merely to show the complexity of the problem of determining the value of a water right. If development is timely and economically sound, its value—in excess of such part thereof as may be based on cost—is so merged with the value of all other intangible elements that separation is necessarily more or less arbitrary. Personal judgment of experts in each particular case must determine the allocation of the aggregate of all intangible values, which, moreover, is rarely of material significance.

JOHN E. FIELD,* M. AM. SOC. C. E. (by letter).—The writer regrets that the discussers have interpreted the purpose and scope of the paper as being much broader than was intended. There was no thought of discussing water-right values in the humid region, but in the semi-arid West, where the riparian doctrine had been abrogated and the water was subject to appropriation for beneficial purposes, and where, because the supply was less than the demand, values were created.

Mr. Jacobs concludes that water rights should be included in appraisals, and the writer agrees with him. While in theory the water may belong to the State or the public, the right to its use is a property right. The carrier has a right to the continued carriage and may charge for and derive revenue as a consequence.

If the water right has been acquired by purchase there can be no question of the propriety of including it in an appraisal. In places where it has been filed upon for the use of others (as in a municipal water system belonging to a public service company), it is sometimes contended that, since it belongs to the users—the citizens—such a right should not be included in an appraisal of the company's property; but, in Colorado, the Courts have held that it should be included because the company has the right of carriage. After the point of diversion has been fixed and the quantity and date of priority have been determined, the city cannot change the point of diversion to its own distribution system and deprive the company of its means of revenue. Under the ruling mentioned the value is listed under "carriage right" instead of "water right", and the final result is little changed.

If water that is otherwise running to waste is impounded, it becomes in effect private property. Water that is carried through tunnels or otherwise from one water-shed to another (as is permitted under the decision of the U. S. Supreme Court in the case of Wyoming *vs.* Colorado), is so under control that it, too, is in effect private property. Engineers may safely assume the propriety of including water-right value in general appraisals.

There appears to be one fundamental difference between the writer and most of the discussers in the conception of a water right. Is a water right a tangible or an intangible thing? If it is tangible, then all comment of its inclusion in an appraisal and the necessity of separating it from intangibles, as shown by both Mr. Grunsky and Mr. Jacobs, disappears. Appraisal of water-right values would be much simplified if, instead of using the term "water-right", one used the word "water" only.

The right to use water is tantamount to its ownership; it should be treated as a physical thing with a value to be established the same as for the physical plant. Often the desire is to obtain the actual benefit derived and the actual value to the owner and such value continues until abandonment, which is not to be contemplated as a possibility in most cases any more than the abandonment of the physical works. For example, in Colorado, many smelters and mills which used water for cooling and other purposes, were abandoned. The value of the physical plant then became nil while the value of the right to the use of the water remained almost undiminished. It could be transferred and used elsewhere for similar, or under certain conditions, quite different purposes. Abandonment through non-use is the only way the right is lost and even then it is a question of intention. The abandoned beet sugar factories furnish another example of the permanence of the value of a water right.

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* Denver, Colo.

In the case of the smelters, had an appraisal been made before 1893—the year of the demonetization of silver and the beginning of the decline of mining—and had the water been considered as merely a franchise and for that reason a small value was given it, then it would seem that the appraisal of the physical plant should have taken possible abandonment into account, for even then the possibility of the demonetization of silver was under discussion. The abandonment of the water right or its forfeiture or cancellation was, in fact, less probable than abandonment of the physical plant.

The writer's suggestion is that the appraiser get at the truth—the real value—first, and then, as far as the necessities of Court decisions require, alter the nomenclature and form of the appraisal to conform to precedent and let the Court alter any parts that may be in doubt.

In regions where water is so thoroughly in the control of the user as it is in Colorado, where it may be bought, sold, transferred, and its use extended and changed, the conception that water is the product that is owned rather than a water right is thoroughly fixed in the lay mind. While Courts and engineers have discussed "rights" in an academic way, in the final analyses they, too, have treated it as a physical thing. In States where the doctrine of riparian rights has been set aside, and water ownership has been vested in the public, the fact that appropriation to beneficial use "shall never be denied", makes the right a very ponderable thing. The act of appropriation for beneficial use and compliance with the regulations pertaining thereto result in removing the ownership from the public to the individual and only the reversion right remains in the public. Right-of-way deeds and grants for railroads generally contain a reversion clause and the value of the right of way is allowed in appraisals to the full extent of actual use and necessity; the value is based on that of similar land in the vicinity. The right of way and the water right in these reversion features are similar.

The propriety of including an indebtedness of $600 000 in the value of the Fort Lyons Canal has been questioned. The value of the whole property, as determined by taking the market value of a share and multiplying by the number of shares, was $6 000 000, and this was verified by the President of the Company who also informed the appraiser that there was an indebtedness of $600 000 which was being paid off gradually by assessments on the shares. The total value when the debt was paid, in theory at least, would be $600 000 more than it was before.

More details of the method by which the $2 000 000 value of the physical plant was obtained have been requested. There was available the original cost of the work, including interest during construction, engineering, rights of way, etc. Relative costs of labor and material as of date of construction and of appraisal were ascertained. An examination on the ground was made; the cost of the new structures which had replaced the old ones was determined; and a value was added by reason of the seasoning of the canal, based on an examination of repairs and upkeep during the first years of operation. Indeed, in this case, the cost of maintenance was at hand for each year since the canal was built. From all these items the cost of replacement was obtained, a small sum was added as value of assemblage, bringing the amount to an even $2 000 000. It was not the Fort Lyon Canal that was being appraised. The appraisal was not a rate-fixing case, nor one to be reviewed by any Court. Its purpose was to ascertain the value of the many water rights of the Company as nearly as possible and approximately correct as a whole rather than as to details; additional expense was not incurred in unnecessary refinements.

Furthermore, the water-right values of three canals (the Fort Lyon and two others) were taken and these were selected for their simplicity of ownership, capitalization, stability of stock values, success in operation, conservatism in management, and progressiveness in improving the character of structures, etc. In short, three of the best managed and most successful canals were selected and the value of a number of water rights of different priorities was obtained and plotted in Fig. 1. With this curve the probable value of any priority in the same water-shed could be "picked off"; and with this probable value as a base from which to work, the values of the numerous canals, pipe lines, domestic supplies, etc., embraced in the appraisal, were determined.

In several cases this probable value was changed as affected by use, soil, locality, abundance or scarcity of water, supply and demand, yield, etc. In fact, the major part of the work of appraisal was not in determining the base curve, but in applying needed corrections. The base curve was valuable, enabling one to show a consistency in results, or good reasons for an apparent inconsistency.

Mr. Jacobs lists numerous factors that have a bearing on water-right valuation. An example of Factor (*a*) is the difference between the laws of Wyoming and Colorado relative to water being attached to the land. In Wyoming, water may not be transferred and through more economical use, it may not be made to cover a greater area than the permit shows. Except as noted later the title to water goes with the land; its use is comparatively rigid and inflexible. In Colorado, on the other hand, its locus of use may be changed even to diverting it into another canal. It may be made to cover a greater area and, therefore, it is an article which may be sold, the value being largely determined by supply and demand.

Referring to Factor (*b*), in some States there is an irrigation season. In Colorado, the only time limit is the needs of the appropriator, beneficial use being the limitation of the right. There are, also, canals in which water flows during all but the most severe freezing weather. Moreover, water may be temporarily stored to be used later in the season. These extra privileges have value and affect the appraisal.

In States that operate under the Wyoming Code, or modifications thereof, the separating of the water values and land values is not generally necessary; but, in Colorado, the water right must be specifically named in the deed to the land if the intention is to let it go with the land.

The writer approves of Factors (*c*) to (*g*), inclusive, with the supplementary comments by Mr. Jacobs. As to Factors (*h*) and (*i*), it might be added that agricultural uses furnish the alternatives for a domestic supply, and agricultural values influence strongly, if they do not actually fix, the value of a domestic supply. Factor (*j*) appears to be highly improper when the object of the appraisal is rate fixing. It would not be proper, except for purposes of comparison with other methods, when the value could be found by any other method. The process in this case is merely to start with an answer and work back.

Mr. Jacobs and others call attention to the writer's statement that "whatever amount was paid for * * * old works that were abandoned or destroyed, was added to the cost of the water or water right." The water right was purchased with the knowledge that the old works would not be used, and as far as the purchaser was concerned, the amount he could afford to pay was for the water right only. The seller doubtless included the value of the old works in estimating for what he could afford to sell; but the water right was the only thing of value to the purchaser, so that, in appraising the new works, the value of the water right to the new system of necessity includes the seller's estimate of value of the old works which, at the time of appraisal, are nonexistent.

It seems that several discussers have confused the value of water stock or shares in a corporation and the value of a water right as used by the writer. The first is the value of shares as shown by numerous sales over a period of years. It is the market value of the shares. The second is the value which is paid for water without physical works and for water to be transferred to some other system. It is the value of a surplus supply owned by some one anxious to sell, and which he can no longer use with reasonable profit; it is a right which, from some cause, he is compelled to sell, possibly from the danger of abandonment and of forfeit; possibly the price is greater than the nature of his own lands and crops justify him in holding.

Mr. Jacobs states: "As applied to private industry, the logic of capitalizing excess net earnings as a basis for determining water-right values can hardly be questioned." Such a method may be logical, but it might be neither just nor correct. Values as determined by actual sales, by the cost of alternative supplies, and by the values in similar enterprises and conditions, might all point clearly to the unreasonableness of an appraisal on such a basis. In cases where all these possibilities of using other methods are absent and there is no power of rate fixing, then such a method of necessity might be used, not as really fixing the value, but as an element in guiding the judgment toward a reasonable valuation. Instead of raising the value of the water right beyond a reasonable figure, a large interest return on the investment as a whole should be shown.

The writer once made an appraisal of water which was used for condensing purposes and was immediately returned to the river. The saving in coal, operating cost, and investment was cut about 33% below the estimated cost of non-condensing engines. Underground water could have been pumped at a relatively small expense; this right of use could have been abandoned and another filing of almost equal efficiency could have been made. The net earnings of the mill, through the use of the water, were enormously increased, making successful competition possible; but the value of the water right in itself was small.

The writer agrees with Mr. Jacobs that "no single method of valuation can be rigidly applied * * *". In all cases all possible methods should be used; their importance and applicability should be weighted, and the final conclusion determined in accordance with the appraiser's judgment.

In a number of the discussions the writer's statement that "stored water" has little more value than "unstored water" has been criticized. The writer should have said that "water available for storage" has little more value than that available for direct diversion and use of the same date of priority. Much stored water is sold year by year in Colorado, not always to the same user, but to any one who has need of it. The price is fixed by the supply and demand at the time of sale. In a year of large flow, it has little value. Generally, the seller insists on a reasonable return on his investment in the physical works over a period of years, while the purchaser considers the value to him as an element that helps to grow, save, or increase his crops. When the average charge is great or becomes unreasonable, the use is curtailed by more careful handling and avoidance of waste, or by the construction of additional storage facilities. Such cost at the time of construction is almost the full value of the water, leaving little or nothing as the value of the water right. The gradual advance in values and of returns on such an enterprise accrues to the value of the right.

Mr. Tibbetts states that: "The few public utilities serving water in California seldom have a total stock value equal to that of their appraised physical properties * * * although it is recognized that their water rights have substantial values * * *". He does not state whether such utilities have mortgages, bonds, or other debts outstanding. If there is indebtedness, then it should be added to the stock value as the writer did in the Fort Lyons case, and, by the method proposed, it might be found that the water right had a value. There are enterprises, the physical costs of which were more than the value of the water delivered, as in some of the reclamation projects. This excess of cost over value must be recognized by the appraiser, and corrections made. Such a project should not be taken as a basis for determining values or rights on other projects. On the other hand, if a project, such as this, is being appraised, then the method proposed in the paper appears to be eminently suitable. The values of the water right would be determined by examining other projects, and these need not be confined to public utilities, but private, municipally owned, and mutual projects could be used as a basis for constructing a curve of water-right values. The result on a project such as that mentioned by Mr. Tibbetts would show that the investment in physical works was too great for the obtainable revenue and also the amount of the excess.

Mr. Tibbetts states that, "if he were to purchase a large amount of stock * * * he would first appraise the physical properties and * * * water rights * * *". That is exactly what the writer proposed. He ascertained the values in other projects and applied those values to the properties being appraised.

Mr. Tibbetts "sees no reason why the value of water for domestic purposes should not be based on the city's necessities and on what the city can afford to pay." The reasons are that agricultural water can be purchased for domestic use (as was done in Owens Valley) at agricultural prices, and that there is no practical limit to what a city can afford to pay. Mr. Tibbetts in this connection cites Southern California. If agriculture by irrigation in Southern California should absolutely cease, there would be no need to bring water from great distances for domestic use, and the demand for such use would dwindle with the dwindling population.

Mr. Tibbetts' remarks, under "Land Increment Value", are particularly pertinent. A bonus to the land owner should be included in "land increment value". At this time in particular it is necessary at least to hold out the hope to the man who takes a raw piece of land and develops it into a going concern, that he will receive a substantial advance in the land value over and above cost and labor. It is the task of determining the size of this bonus that makes land increment value difficult as a basis for determining water values and as a means of allocating the difference in value of adjacent unirrigated land and the irrigated land, to the land, the water, and the bonus. Mr. Hedke calls this "margin".

In Fig. 2, Mr. Tibbetts shows the average yield and an assumed water demand. It should be recognized that the yield curve determines in large measure the demand curve, because with a given yield curve the user adjusts the character of his crops, methods of farming, type of physical irrigation works, and the acreage irrigated to conform approximately to the yield curve. It is almost invariably true that the yield and demand curves are similar, the difference being more in position than shape, because the demand curve is later in point of time. The construction of reservoirs is the means generally used to make the demand curve and the supply curve more nearly coincident.

Referring to Mr. Scoby's discussion, it is well recognized that shares, stock, rights, and even measurements of water, such as the inch, vary widely in meaning. To avoid confusion, the writer adopted volume delivered as the measure, and he thought he had avoided the necessity of using the variable time element of an irrigation season by adopting the full year as the theoretical maximum. The use of the full year could not have been avoided because many of the rights to be appraised are used for 365 days. Had some arbitrary irrigation season been adopted, such as 180 days, it would have been necessary to give some of the properties a delivery coefficient of 200%, and the meaning of 200 per 100 is paradoxical, confusing, unnecessary, and complicated.

Mr. Hedke states that "it is natural, therefore, that Mr. Field * * * should complacently open an unsettled subject without ever mentioning ownership." The writer admits also to complacently assuming that values of water rights should be included in appraisals, that they have value, that the user to all intents owns the water, and that they are susceptible of appraisal by fairly rational means, simple in principle, but difficult in application.

Mr. Grunsky states that efforts to find a simple, generally applicable method will be in vain, the main difficulty being in allocating the intangible values to the individual elements. This is true; all appraisals are difficult, but they are based fundamentally on the physical plant which can be appraised in a rational way. To the writer a water right appears to be a physical element, a tangible and not an intangible thing.

Mr. Stivers discusses interestingly various methods of appraisal and the Court decisions thereon. He states that the method proposed in the paper is unsound, because it is not a generally accepted method. Any method, however new, untried, or even revolutionary, if it is reasonable, and is evolved in a rational way, will doubtless be accepted by the Courts. It is the duty of the appraisal engineer to use a rational method regardless of other and "generally accepted" methods. The Courts have modified, changed, and even reversed previous opinions too often to cause an engineer to hesitate to use a method he believes to be good. Indeed, the use of a method that is questionable with other methods for comparison and guidance to final conclusions, is advisable. That engineer is wise who studies Court decisions. Through them he obtains ideas and "food for thought"; he finds methods analyzed and learns the reasons for using them; they aid him in developing the judicial attitude; but to say he is bound by such decisions—that he must accept them as final—would be to stop progress in the evolution toward better methods.

Mr. Stivers has given an interesting account of an appraisal of a water right owned by a railroad company in New Mexico, and has used a rational method for its determination. When based, as it is, on an assumed duty of water and on a stated difference in land values before and after the land was deprived of water, the method is correct. The analysis of the result is confined to the correctness of the determination of the duty of water and of the difference in the land values.

It seems that the value of the non-irrigation season flow should have been specifically determined by ascertaining the cost of storage for agricultural purposes, and whether or not such an expense was justified. Deductions in the quantity stored, by reason of evaporation, seepage, and other losses, should be made, and the availability of land and its return in crop value should be studied. The amount for which the railroad company could have sold the water to some other users, and what those users could afford to pay, would fix the value of the winter flow more certainly than if based on the value of the irrigation season supply as was done by Mr. Stivers. If this estimated sale price of the stored water exceeded a reasonable return on the investment plus the cost of maintenance and operation, then the winter flow would have a value, and the amount of the excess earnings would be the basis of that value.

In referring to the temporary use of the water, Mr. Stivers states that the farm operations of the railroad do not pay. If the same result should obtain under good management by others (farming undertaken by a corporation as a side line, and as a matter of policy and necessity, seldom pays), then the values given to the winter flow are erroneous unless justified by values obtained by some other method, and some demand other than agricultural.

In conclusion, it appears to the writer that, while the laws governing water rights vary in the different States, their customs and decisions are more nearly in harmony. In Wyoming, where the Code differs radically from that in Colorado, the Court decisions are generally in harmony with, if not actually based on, earlier Colorado decisions. Many water rights in Wyoming were acquired under territorial laws. These, in particular, have been treated as, and have a standing similar to, rights in Colorado. The tendency in Nebraska, Kansas, Wyoming, and New Mexico is to follow the Colorado decisions. Practice in Colorado is based largely on decisions, which cover almost every phase of every water-right question. They have undergone the test of time; they have been modified when advisable, and, therefore, they present to other Courts a well established and practical solution of most of these questions. While examples and cases applicable particularly to Colorado have been used in argument and illustration herein, they are believed to be largely applicable to other States.

All those who have discussed the paper have shown unusual familiarity with water, its uses, and the problems involved in considering, not only values, but uses, customs, laws and practice.

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Paper No. 1729

ADMINISTRATIVE WATER PROBLEMS

A SYMPOSIUM*

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* Presented at the meeting of the Irrigation Division, at Denver, Colo., July 14, 1927.

TRANSMISSION AND DELIVERY OF RESERVOIR WATER

By G. Clyde Baldwin,* M. Am. Soc. C. E.

Throughout the early stages of irrigation development in the West reservoirs were of little consequence. Land was watered by direct diversion from the natural streams with small effort and at little expense. The success of the pioneers naturally attracted others; more difficult enterprises were undertaken; and following the enactment of the Federal Reclamation and Carey Acts, really large-scale undertakings were initiated. By this time, also, the low-water flow of many of the streams had proved to be insufficient to supply all demands. For this reason, and because of the feasibility, under these laws, of financing the construction of dams and other necessary works of considerable magnitude, storage reservoirs began to play a part of constantly increasing importance in all reclamation plans; until, to-day, they are usually considered to be the foundations upon which most new irrigation projects must be based.

Low unit storage cost, together with the physical possibility of making gravity deliveries, chiefly governed the location of these early reservoirs. Scant consideration was given to the other difficulties which might later be experienced in connection with their operation.

Where the selected impounding basin is located close to the irrigable lands, on otherwise dry channels, or on small streams where no conflict with other water rights exists, these operation or delivery problems are comparatively unimportant. They become very real and worthy of consideration, however, where the storage must be transmitted for long distances down the natural channel of a large stream, most of the normal flow of which has been awarded to other and earlier appropriators.

Situations of this character are now to be found in most of the Western States, but probably none is more complicated or involves the administration of such large quantities of water as that of the Upper Snake River Basin in Idaho and Wyoming. In that portion of this area which, for water distribution purposes, is administered as a single unit, four major and several minor storage reservoirs are in operation. Without taking into consideration small diversions on the upper reaches of the streams, which are relatively isolated and where net consumption of water is usually small, water is diverted from the river and its larger tributaries through about 120 principal canals.

The four large reservoirs are Jackson Lake, situated at the upper end of Jackson Hole, Wyoming, and only a few miles south of Yellowstone National Park, with a capacity of 847 000 acre-ft.; Henrys Lake, near the head-waters of the North or Henrys Fork of Snake River, the capacity of which is about 75 000 acre-ft.; American Falls, on the river just above the Town of American Falls, Idaho, where the full capacity of 1 700 000 acre-ft. is available for use this year (1927), the first since the completion of the dam; and Lake Walcott, created by the Minidoka diversion and power dam of the U. S. Bureau of Reclamation, where about 107 000 acre-ft. is normally impounded and can be used, at the sacrifice of power head, in times of urgent need.

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The canals range in size from those with a capacity of only 1 or 2 sec-ft. up to one of more than 3 600 sec-ft. The maximum for the entire group amounts to nearly 30 000 sec-ft., while the total diversions from May 1 to September 30 in a normal water supply year equal almost 6 000 000 acre-ft.

Henrys Lake Reservoir is owned and utilized entirely by canals in the vicinity of St. Anthony, Idaho, while Lake Walcott belongs to the adjacent Minidoka Project. Roughly, 15% of the combined storage impounded in Jackson Lake and American Falls Reservoirs belongs to canals in the so-called Idaho Falls Section; 14% to the Minidoka Project; 38% reserved for as yet undeveloped enterprises; 2% to the Idaho Power Company; and the remainder, or 31%, to the Twin Falls-Jerome Canals which divert at Milner Dam, the extreme lower end of State Water District No. 36.

The natural stream channels must be used for carrying all stored water, except that impounded in Lake Walcott, from the reservoirs to the points of final diversion and use. In the case of Jackson Lake water, this means transmission of more than one-half the total volume for a distance of about 300 miles, past the head-gates of the Idaho Falls region canals, and on through the American Falls and Lake Walcott Reservoirs.

Normal flow water rights have been decreed at the canal head-gates, but stored water, on the other hand, is acquired at the reservoir. Both the States of Wyoming and Idaho provide by statute for the use of stream beds as channels for the conveyance of stored water. However, there is no prescribed basis for converting a reservoir right into a right at the point of diversion many miles down stream. Every one agrees that some deductions should be made for transmission loss, but there is a wide divergence in views as to how this loss shall be determined. This ranges from a minimum proportional charge based on estimated additional evaporation and seepage resulting, respectively, from the extra exposed surface and wetted perimeter caused by the storage run, up to a direct proportional quantity tax through each small section of river channel where net losses are known to occur, with no corresponding credits in sections where the river shows a net gain.

During the first few years after the completion of the original temporary dam at the outlet of Jackson Lake, attempts were made to deliver the storage to the lower valley in a series of flush heads. Prior to the arrival of each of these flushes, canals in the Idaho Falls area were staked, and river riders were employed to make certain that this normal draft was not exceeded during the storage run. Under this delivery plan Lake Walcott was operated as a catch-basin and equalizing reservoir. This method resulted in net storage losses amounting to between 25 and 30%, while the alternate high and low river stages were very destructive to the less permanent diversion dams along the stream and made difficult the maintenance of any uniform flow in the canals. Consequently, it was unsatisfactory alike to stored water and to normal flow users. Beginning in 1912 this method was abandoned in favor of a continuous flow system which, with certain modifications pertaining chiefly to transmission losses, has been in use since that time.

Under the latter system the rate at which storage is released from the reservoir varies with the needs of the stored water users, but abrupt fluctuations of any magnitude are avoided. Allowance is made for time of transmission and loss in transit. Then, the intervening canals having normal flow rights are regulated in accordance with the decrees to insure proper deliveries each day. The daily determination and frequent regulation of flow in each canal, as well as the maintenance of continuous records at numerous river points, are required.

For many years, the amount to be charged against this storage as transmission loss continued to be subject to dispute and was provocative of much dissension as to stored and normal flow, or on geographical lines, between lower and upper valley water users.

Numerous partial investigations of different phases of this problem were authorized from time to time. Most of these, because of the experienced and disinterested personnel obtainable, were conducted by, or under, the supervision of the Water Resources Branch of the U. S. Geological Survey. These investigations served gradually to accumulate certain facts concerning river losses and gains and the reasons therefor. The problem is an extremely complicated one, however, involving many variable quantities, so that even if much more money were expended and the investigation made comprehensive over a period of years there appears to be some doubt as to whether a transmission loss schedule can be evolved, which will be absolutely equitable under all conditions.

Meanwhile, deliveries have been effected from year to year under what might be termed compromise schedules, based partly on investigation or fact and partly on mutual concession by the different classes or groups of users. Actual applied losses from Jackson Lake to Blackfoot have ranged from 5% in some of the earlier years to about 15%, with the average for recent years about 13 per cent. These are subdivided as follows: From the outlet of Jackson Lake to Heise, Idaho (canyon section), 2½%; from Heise to Lorenzo (so-called Snake River cone), actual percentage loss each day as determined from the average of the two preceding days' records, with a minimum charge of 3%; from Lorenzo to Woodville (section where a net gain in the river is usually noted), ½%; from Woodville to Blackfoot (heavy loss section), 6 per cent.

Another troublesome item is the equitable segregation of stored water from normal flow at the outlets of the respective reservoirs. For Jackson and Henrys Lakes the practice of relying chiefly on the reservoir capacity tables for the determination of the amount of storage released daily, has been followed for a number of years. Special investigations indicated that while this method does not accurately apportion all the factors that make up the total outflow at all times and under all conditions, it is, in general, a reasonably fair plan of distribution and has the advantage of being easily operated. This method automatically credits the normal flow with all increments from bank storage return in lieu of the old natural lake storage and the extra water-surface evaporation.

The situation at American Falls is somewhat different. Here, there was no initial lake prior to the construction of the reservoir and little is known as yet concerning the relative values of such items as deep percolation loss and bank storage return, while the stored water will be subject to an evaporation loss charge for practically the entire exposed water surface. This reservoir is situated in a section where the river previously showed a net gain from large springs and ground-water sources. Data obtained by the U. S. Bureau of Reclamation during the summers of 1924, 1925, and 1926, suffice to determine that a little more than 50% of this gain comes from surface streams which may be gauged even under full reservoir conditions and that the total inflow or gain through the reservoir is quite consistent in its fluctuations with the changes noted in the measurable half.

The normal flow which should pass the outlet gates can be approximated, therefore, by adding to the water which enters the reservoir through the main river channel the product of the daily measured gain and the predetermined coefficient. This method is both laborious and expensive because it requires daily measurement and summation of the flow of about twenty-five rather inaccessible streams. Furthermore, there is no assurance that a coefficient which was applicable in one year will also apply in another; but it affords a determination that is independent of evaporation, percolation, bank storage return, and other such items, all of which were studied during the summer of 1927 with the idea of evolving a simpler procedure for the future.

The use of space in Jackson Lake Reservoir for the storage of decreed water rights and the later release and delivery of the water thus held back were authorized by mutual consent of all water users in 1924 and again on a more restricted basis in 1926. This plan encouraged rotational use of water and undoubtedly helped to promote higher duty usage during those years when the total supply was deficient. Its adoption adds many complications to the work of water administration and because of the departure from normal procedure the plan is perhaps subject to attack on the question of damage.

The human relations feature of the water distribution work is probably more important than any other phase. It is often said that in an irrigated country an otherwise upright man will not hesitate in times of drouth to steal water and that the most peaceable citizen under ordinary conditions may quickly become one of the most violent if he thinks his water rights are being attacked or infringed in any way. Certainly, water disputes are easily started and sometimes become very bitter.

In this respect the Upper Snake area is not materially different from other sections; but there appears to be a growing tendency to try to adjust these disputes without violence and without recourse to long drawn out litigation. This attitude is believed to be largely attributable to a so-called "Committee of Nine" the members of which are elected each year by the water users to act as advisers to the district water master. At any rate this Committee has been most helpful in making contacts between the administrative officer and the individual water users. It passes on the annual distribution expense budget and on all matters of policy that are not clearly defined or established. In many other respects it may be compared to a Board of Directors for the Water District, of which the Water Master, or Deputy Commissioner of Reclamation, is the General Manager.

The writer has held the latter position, as well as that of Special Deputy in charge of stored water delivery, since May, 1919. In addition, he has acted as District Engineer for the U. S. Geological Survey in charge of hydrometric work within the same area. In March, 1923, and, subsequently, each year he has also been elected to the position of Water Master, thus further centralizing the water distribution administration. A skeleton organization consisting, in addition, of two engineer assistants and a clerk, is maintained throughout the entire year while deputy water masters, hydrographers, and other employees are added as needed during the summer months. A limited edition of a special report of each year's operations is prepared during the winter while stream-flow data are also made available through U. S. Geological Survey Water Supply Papers and the biennial reports of the State Commissioner of Reclamation.

While of necessity details have been largely omitted in this outline and many important phases of the work have been touched only briefly, it is sincerely hoped that this procedure, which has stood the test of many years' actual operation, may be of value in other sections where similar problems are encountered, or where they may develop with more complete water usage.

ADMINISTRATION OF STREAM FLOW

By Edward Hyatt,* M. Am. Soc. C. E.

Synopsis

In this discussion of the administration of water supplies under State control in the seventeen irrigation States, special reference is made to California. The conflict between the opposing legal principles of riparian and appropriative water rights has retarded and penalized water development in that State. Efforts are under way to ameliorate this condition.

Comparison of Water Codes

In seventeen Western States irrigation is generally practiced. These include Texas, Oklahoma, Kansas, Nebraska, North Dakota, South Dakota, and all States west thereof. Of these, all but Kansas, South Dakota, and Montana have adopted water codes providing for State administration of the distribution of the waters of their stream systems through the agency of water masters or water commissioners. While the fourteen codes adopted show considerable variance in nomenclature and in points of procedure an analysis reveals substantially the same underlying purpose in each, which is to make possible effective State control of the distribution of water.

Each code provides for the division of the State into water districts. In Colorado and Texas this division has been arbitrarily made by legislative enactment and, in all other States, the water districts are created as needed by the State Water Commissioner, State Engineer, or the official in charge of water matters. In three States the water masters are appointed by the Governor; in one State the water users elect the water masters; and in all others they are appointed by the chief water official. In eleven States the water master has the power of arrest and, in ten, interference with his acts constitutes a misdemeanor. Twelve of the fourteen codes give the water user the right of appeal from the actions of the water master; seven to the Superior, District, or Circuit Court, according to the judicial structure of the States, and five to the State office in charge. In three States the compensation of water masters is fixed by the Legislature, in one by the water users, in one by the county commissioner, and in the remaining nine by the State office.

Financing the work of distribution is accomplished in various ways. In one State—Nebraska—it is paid from general State funds and, in five others, from general county funds. In 1927 the California Legislature passed an act by which half the cost will be met by the State and half by the water users. In the remaining seven States the charges are met in different ways, but in each case are later assessed back against the actual water users. The basis of apportionment of cost among the water users is, in most cases, the respective quantities of water delivered. In one State—Nevada—it is on the basis of the established water rights.

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The methods of administration for the various States are listed in Table 1.

California Procedure

According to the 1920 Census, the irrigated area in California exceeded that in any other State and was more than double that of any other State except Colorado and Idaho. With the statutory water codes approximately the same in all the States and with the greatest irrigated area in California, it would naturally be expected that California would be found in the lead, or nearly so, in the matter of efficient handling of its water resources, particularly in the matter of distribution. This, however, is far from being the case. In both adjudication and distribution California is far behind such States as Colorado, Wyoming, Idaho, and others. The reasons for this condition are not far to seek. The principal obstacle lies in the recognition by California of the riparian principle.

Riparian Rights

Apparently, most of the inter-mountain States have escaped the riparian rights doctrine and its consequences entirely. This rule has caused no end of trouble, and has been a great handicap to California. Years of effort and millions of dollars that should have been put into development have been expended in conflict and litigation. A general uncertainty has been thrown about the legality of water rights, which has added greatly to the difficulties in the way of water projects.

A riparian right rests on land ownership and inheres in land bordering on a stream of water. The right as defined by the California Courts does not depend on use, nor does it cease with disuse; it is superior to an appropriative right except in special instances and is not limited to reasonable use as against the appropriator. It is contrary both in theory and practice to the doctrine of appropriation of water, which rests entirely on use and is, of course, opposed to any general water conservation measures such as have to be resorted to in the West.

Historical

California's water law history has been unfortunate but interesting. The appropriative doctrine was established in the "Fifties" by the miners as a necessity of the time and has been recognized by the Courts from that time on, although not until 1913 was it clearly set forth by statute. The riparian theory was adopted by reference when the common law of England was made the rule of decision in the State by statute of 1850.

The fact that there were two doctrines of water law, contrary in principle, naturally led to many disputes. Neither being adequately set forth by statute such disputes could be settled only by Court decisions. Therefore, in sixty years, about seven hundred State Supreme and Appellate Court decisions on water matters have been given. Covering a range of climatic conditions from humid to arid and such a great length of time, the decisions are naturally overlapping and conflicting; however, they are in the main not subject of change by legislation, and to a considerable degree constitute the water law of the State. It is of interest that the other irrigation States likewise adopted the common law of England, but that nearly all their Courts followed the lead of Colorado and abrogated the common law provision on the general ground that it was unsuited to the needs of the State. If the California Courts had adopted this same attitude the State would have avoided this difficulty.

CALIFORNIA WATER CODE

On account of conflict between the riparian and appropriative factions it was impossible for many years to enact even an appropriative code, but in 1913 this was finally forced through for the initiation of appropriative rights. In 1917 an adjudication procedure was added and in 1921 the distribution feature. The distribution statute was incomplete in that compensation for water masters was not covered, and it was not until the 1927 Session of the California Legislature that the distribution procedure was made workable by the addition of a compensation provision. This law took effect about August 1, 1927.

ADJUDICATION METHODS

Under the 1917 amendment two methods of adjudication of water rights were provided: First, a complete procedure for the adjudication of appropriative rights patterned after the Oregon Act, which was thought to be the best then in existence; and the second authorizing judges of Superior Courts to transfer water litigation to the Division of Water Rights for investigation as referee. The results under these two modes have not been as anticipated. The adjudication procedure provided in the Act is a complete and admirable one and effective in a case involving only appropriative rights; but when the *bete noire* of riparian rights enters the picture the situation is changed. This method cannot determine riparian rights, and, as any adjudication, to be satisfactory, should take account of all rights, it is apparent that this type of action will fail where riparian rights of magnitude exist.

To its sorrow California has found that they exist almost everywhere. A good attorney can develop for a client riparian rights which the client himself would never have thought of claiming. Mainly on this account in the last few years most of the adjudications have been handled through the Court reference method. To accomplish this a suit is filed, sometimes a friendly suit, and then by stipulation the case is referred to the Division of Water Rights. In this way rights of all classes—appropriative, riparian, and prescriptive—are included, because the Court has jurisdiction over all of them, and the Division of Water Rights in acting as the Court's referee can properly take account of them. After making its investigations the Division reports back to the Court, often presents a suggested form of decree, and the desired result, that is, a workable adjudication of rights, is accomplished.

This method is not without its difficulties, however, as it is usually necessary to re-frame the pleadings to the suit, so that all parties cross com-

TABLE 1.—ADMINISTRATION OF STREAM SYSTEMS BY WATER MASTERS, AS OF JULY 1, 1926.

State.	Divided into water districts by:	Water masters appointed by:	Water masters supervised by:	Water master has power to arrest.	Penalty for interference with regulation by water master.	Water users have right of appeal to:
Arizona	State Water Commissioner.	State Water Commissioner.	State Water Commissioner.	Yes.	Misdemeanor.	Superior Court.
California	Chief of Division of Water Rights.	Chief of Division of Water Rights.	Chief of Division of Water Rights.	Yes.	Misdemeanor.	Superior Court.
Colorado	Legislature (State divided into 70 Water Districts.)	Governor.	State Engineer (through Division Engineers).	Yes.	Misdemeanor.	State Engineer (through Division Engineers).
Idaho	Department of Reclamation.	Elected by water users.	Department of Reclamation.	Yes.	Misdemeanor.	Department of Reclamation.
Nebraska	State Department of Public Works.	State Department of Public Works.	State Department of Public Works.	No provision.	No provision.	No provision.
Nevada	State Engineer.	Governor.	State Engineer.	Yes.	Misdemeanor.	No provision.
New Mexico	State Engineer.	State Engineer.	State Engineer.	No provision.	No provision.	State Engineer.
North Dakota	State Engineer.	State Engineer.	State Engineer.	Yes.	Misdemeanor.	State Engineer.
Oklahoma	State Engineer.	State Engineer.	State Engineer.	Yes.	Misdemeanor.	State Engineer.
Oregon	State Engineer.	State Engineer.	State Engineer.	Yes.	Misdemeanor.	Circuit Court.
Texas	Legislature (each county a water district).	State Board of Water Engineers.	State Board of Water Engineers.	No provision.	No provision.	District Court.
Utah	State Engineer.	State Engineer.	State Engineer.	Yes.	No provision.	District Court.
Washington	State Supervisor of Hydraulics.	State Supervisor of Hydraulics.	State Supervisor of Hydraulics.	Yes.	Misdemeanor.	Superior Court.
Wyoming	State Board of Control	Governor.	State Board of Control.	Yes.	Misdemeanor.	District Court.

TABLE 1.—(*Continued.*)

State.	Compensation of water masters fixed by:	Water masters paid by:	Funds from which water masters are paid.	Method of collection of funds.	Basis of apportionment of cost among water users.
Arizona	State Water Commissioner.	Water users.	Assessments ordered by Superior Court each month.	No provision.	Discretionary with Superior Court.
California	No provisions.	No provision.	No provision.	No provision.	No provision.
Colorado	Legislature.	County.	General County fund.	General County taxes.	Not apportioned.
Idaho	Water users.	County.	Current County expense. fund.	Special County taxes against lands served.	Quantity of water delivered.
Nebraska	Legislature.	State.	General funds of Department Public Works.	General State taxes.	Not apportioned.
Nevada	State Engineer.	County.	County Water District funds.	Special County taxes against lands served.	Established water rights.
New Mexico	State Engineer.	County.	General County fund.	Special County taxes against lands served.	Quantity of water delivered.
North Dakota	State Engineer.	County.	General County fund.	Special County taxes against lands served.	Quantity of water delivered.
Oklahoma	State Engineer.	County.	General County fund.	Special County taxes against lands served.	Quantity of water delivered.
Oregon	State Engineer.	County.	General County fund.	General County taxes.	Not apportioned.
Texas	County Commissioner.	County.	General County fund.	General County taxes.	Not apportioned.
Utah	State Engineer.	State Engineer.	Specified Water Master fund.	Assessments against water users by State Engineers.	Discretionary with State Engineer.
Washington	State Supervisor of Hydraulics.	County.	General County funds.	General County taxes.	Not apportioned.
Wyoming	Legislature.	County.	General County funds.	General County taxes.	Not apportioned.

plain, and to have the Court order into the case all users of water from the stream in question, not already parties to the suit. Therefore, unless the judge and the various attorneys are actively interested in seeing that a true adjudication results, some difficulty is apt to be encountered.

ADJUDICATION RESULTS

Adjudication proceedings on twenty streams in California have been undertaken by the Division of Water Rights, fifteen under Court reference and five under the adjudication procedure provided in the statute. Nine have been completed and eleven are pending. While three stream systems of considerable magnitude, the Stanislaus, Shasta, and Whitewater Rivers, are being adjudicated, the other proceedings have been principally on smaller systems, mainly in the northern part of the State. The increase in activity under the Court reference method has been marked in the last few years.

DISTRIBUTION

The first instance of distribution of water under the provisions of the 1921 Statute occurred in 1922 when the water users on West Carson River, an interstate stream which flows into Nevada, petitioned for and received service in conformity with that law. Since then, as additional adjudications have been completed, the number of water masters has grown to six, besides some assistants. As no method of compensation was provided, it has been necessary for the water users to subscribe voluntarily the greater part of the required funds, the State lending some aid. This method is of course unsatisfactory and has retarded the extension of water master service. As already indicated, this difficulty has now been removed with the adoption of what is believed to be an equitable method of payment for the water masters.

In some instances the necessity for a responsible official in charge of distribution has been so great that agreements have been effected under which a water master could operate. In several cases adjudications were under way but not completed; and in others the complications of the situation were so great that adjudications by any method were deemed practical impossibilities and mutual agreements were reached among the ditch owners, which permitted distribution.

The outstanding example in this latter class is Kings River, the most important irrigating stream in California. Approximately 700 000 acres are irrigated from this source, and the direct flow rights and diversion capacities amount to 10 000 sec-ft., a flow which obtains in the ordinary season for a very few weeks and which is not reached in a dry season. About 150 000 people, practically all dependent on the results of agricultural pursuits, live in the area supplied by the waters of Kings River, showing the importance of the subject.

This river is an excellent example of the confusion caused by the conflict between the two principles of water rights. More than 200 lawsuits had been filed affecting the water rights of Kings River and more than 40 actual Court decrees entered, which, however, were so complicated and impossible of reconciliation that they could not be used as a basis for either adjudication or distribution. After several years of investigation a schedule devised along the lines of past use was suggested by the Division of Water Rights for Kings River and was accepted temporarily by all parties. Under this schedule the water has been successfully distributed for a number of years.

A somewhat similar situation exists in Sacramento Valley. Here, as an additional problem, the low-lying lands around San Francisco Bay are endangered by salt water entering from the sea. Formerly, the flow of Sacramento River was sufficient to keep the salt water back, but with the development of agriculture in the Upper Sacramento Valley this is no longer the case. A water master with limited authority is in charge of Sacramento Valley.

Speaking generally, both adjudication and distribution are in their infancy in California. As the advantages are demonstrated by the examples at hand there is a greater demand for such regulation. If the riparian question could be satisfactorily adjusted there would be, in the writer's opinion, a development along these lines comparable to that which has taken place in many of the other Western States. California has not of course built up an organization similar to that in Colorado, Wyoming, or Idaho. Its water-master efforts thus far have been more or less sporadic and of small consequence compared to other items of the Division of Water Rights' work. Many cases come up as emergencies for short times only and so far the work has not been extensive enough to warrant a separate organization. The time is approaching, however, when this will have to be provided and it is gratifying to have available the experience and precedent of the other States that have studied these problems for so many years.

Herminghaus Decision

Returning to the most important question affecting California water rights, that of riparian rights, the State Supreme Court during the past thirty years had considerably narrowed this principle by limiting riparian areas by insisting, in some cases, on actual need of water. However, in the latest case, that of Herminghaus *vs.* Southern California Edison Company, the Court in a lengthy opinion made a sweeping declaration in favor of riparian owners, stating among other things that the use of the full flow of a river in order to afford the natural diversion of a small part of its waters through overflow was a reasonable riparian use. In the particular case in question this meant that the flood flow of the San Joaquin River, 20 000 sec-ft., could be required for the natural irrigation of the Herminghaus lands while it was admitted that 180 sec-ft. would be sufficient if artificial means of diversion were used. This principle would mean in effect that storage could not be made on a stream where a lower riparian owner demanded the full flow of the stream for the natural irrigation of his lands.

As on its face this decision appeared to make it possible for riparian owners to prevent up-stream storage, it has been widely discussed and solutions of various kinds have been advanced. For the past six years the State of California has been conducting investigations with a view to devising a plan for the most complete utilization of its water resources. The keynote of such a plan is the storage of flood waters, as the natural flow of the streams of the State during the irrigation season is already practically taken up. This decision then became of importance to the State itself in this connection.

The 1927 Legislature actively took up the question and considered many proposed Constitutional amendments designed to clear the situation. After a hard battle in the Legislature one such amendment was passed. It is known as Constitutional Amendment No. 7 and provides that water rights in California shall be limited to such water as shall be reasonably required for the beneficial use to be served and shall not extend to unreasonable use, unreasonable method of use, or unreasonable method of diversion. While this measure is aimed directly at the doctrine declared by the Herminghaus decision, it will be seen that its effect will depend entirely upon the definition of "reasonable" as applied to methods of diversion and use of water. This can doubtless be clarified by subsequent legislation if the amendment is adopted.

The writer can well imagine that to water users in Colorado, Utah, Wyoming, and the other States which have so fully utilized their water resources, it is incomprehensible that the State of California should countenance such a wasteful legal policy as that set forth in the Herminghaus decision.

PRESENT TENDENCIES IN WATER ADMINISTRATION

By George M. Bacon,* M. Am. Soc. C. E.

The discussion of present tendencies in water administration will be confined to the practice and experience of the State of Utah. It is believed that as much value will attach to a specific case as to a general treatment of the subject. By considering a concrete example, and comparing other experiences with it, lessons of value can be drawn as easily as by trying to deduce such lessons from a more general and broad treatment of the subject.

When the pioneers first settled in Utah and commenced to irrigate, the problem of distribution was one of mutual agreement. Later, as the waters of the various streams became fully appropriated, the inevitable litigation set in, resulting in the usual practice of having the Court that decided the rights administer them through a water commissioner of its own appointment. With the spread of irrigation throughout the State, and the creation of the office of State Engineer, it was recognized that many advantages would come through having the administration centered, if possible, under one single authority.

To effect this a statute was passed giving the State Engineer authority, in his discretion, to establish water districts and appoint water commissioners to distribute the waters. There followed, as might be expected, a conflict between the jurisdiction and authority of the Courts and that of the State Engineer in the administration of water rights. This controversy was decided by the Supreme Court of Utah in favor of the State Engineer, who was held to have administrative charge of the distribution of water in the State and who, therefore, could, where the proper formalities had been complied with, supersede the Court Commissioner by one of his own appointment.

Along with this gradual development there grew up the most important theory that the determination of the rights was solely a matter for the Courts, and that the administration of these rights was in the hands of the State Engineer. This division and separation of authority is thoroughly sound and of great practical value in avoiding the overlapping of functions which must always produce confusion.

One of the chief administrative difficulties in Utah arises from the fact that water rights are of two distinct origins. The older rights are the result of appropriation merely by beneficial use and, in a great many cases, are not of record anywhere. Where these rights have been the subject of litigation there followed a Court decree, but these Court decrees are very seldom definite, and some of the older ones are so faulty that they prevent any intelligent distribution of the water according to their terms.

In 1903 the law was passed requiring application to the State Engineer as an essential for the acquirement of right to unappropriated water in Utah. After the passage of this law an additional complication arose, owing to the fact that a few water users scattered over the State, either knowingly or ignorantly, ignored this law and attempted to acquire rights merely by the diversion of water to beneficial use. This situation existed for more than twenty years, until it was finally cleared up by a decision of the Supreme Court of Utah in 1925, which held that no water right initiated subsequent to 1903 was valid unless acquired through application to the office of the State Engineer.

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It is obvious that a full and complete record of water rights initiated subsequent to 1903 exists in the office of the State Engineer; but as to the rights acquired prior to 1903 the record, if any, is scattered through the files of the various District Courts in the form of decrees. A beginning of remedying this unsatisfactory condition was made by the passage of a law in 1919 prescribing a method of adjudicating water rights through co-operation with the office of the State Engineer. In the earlier litigation there was always a great mass of conflicting testimony and no impartial study or investigation of any sort.

The new law is most valuable in that it provides for this impartial study and investigation by the State Engineer's office with a definite recommendation to the Court in the form of a proposed determination as to what, in the judgment of the State Engineer, the rights should be. Ample protection to the water users is secured by a provision that each must be served with a copy of this determination, and that any water user may object to the right proposed for him and have his day in Court to present such evidence as he may desire, showing the incorrectness of the right proposed.

Considerable progress in adjudicating rights has been made under the new law which has proved very satisfactory to the water users in its technical results, and also, in that its cost is but a fraction of the cost of litigation under the old method.

Real success of the administration of water rights is dependent wholly on an exact description and definition of such right. In the various districts where adjudications have taken place under the law of 1919 this information is available, being equally at the disposal of the water commissioner and of the various water users, with the result that the matter is wholly in the open and each user knows just what his neighbor's rights are. This makes for the essential understanding which is necessary for co-operation between all concerned and the old differences and frictions are practically eliminated.

In the older districts where the rights are less clearly defined there are complications not only of proper physical distribution but of the assessment of its cost as well. Continued efforts are necessary to educate the water users in such districts as to the difference between an actual right and the quantity of water available to supply such right.

With practically no exception all the water sources in Utah are heavily over-appropriated, which means that the problem of distribution is entirely different in a dry year from that in a wet year. In a dry year the later appropriations must have their supply cut out as the season of high water draws to an end, and this means that all rights are not of equal value.

Therefore, for the purpose of assessment, the rights are frequently divided into two or three classes and the levy per acre is made considerably less in the case of those rights that fail of supply at the end of the high-water season.

While the fundamental law of water rights, "first in time first in right", is in general controlling, there are a number of exceptions in a dry season. Besides this fundamental law is a secondary division according to use, into domestic, irrigation, power, and miscellaneous rights. The laws of Utah imply that the matter of priority does not necessarily run to the extent of withholding a junior domestic right to supply a senior irrigation right, or a junior irrigation right to supply a senior miscellaneous right. An added complication is found in many of the older Court decrees where the dates or priority were uncertain of record and the Court in its decree divided rights into classes. Where this situation exists it is also usual to deviate from the classes to take into consideration the secondary priority as already outlined.

It is possible that with future developments some method may be found of doing away with the classification in these old Court decrees, at least to the extent of having one fixed early date covering the unknown and uncertain priorities and, after this, definite dates for the other rights. At the time the Utah adjudication law of 1919 was passed there did not seem to be sufficient progressive sentiment to write into it a provision which reviewed in whole these old Court decrees and substituted for their provisions the modern and latest description of water rights. In formulating the State Engineer's proposed determination, it is necessary to follow quite closely the terms of the Court decrees, except where these in actual practice have been found by the irrigators to be so unsatisfactory that they are glad to have them changed.

There is another extremely important phase of administration which can only be mentioned. This has to do with the administration of such water rights as are of record ownership in some agency of the Federal Government. Two types of this situation are the distribution of waters owned by the United States Indian Service or its successors in interest, and water rights and lands embraced in Federal Reclamation Projects. At present (1927), there is not the slightest attempt on the part of the State to take over the Federal control of distribution any more than there is active effort by the State Engineer to supersede by his commissioners the various Court commissioners still distributing water on numerous streams in the State. On the other hand, no good can come, either of division of authority or indefinitness of such authority. In view of the accepted theory that States control water rights and of the tendency, observed from time to time, toward centralization under Federal control, a definite policy must be pursued by States in the arid region with reference to retaining these particular rights which have been regarded as definitely settled by decision of the United States Supreme Court in favor of the State. If attempts to view this matter as not finally settled are ignored, there is always the possibility that the question might be re-opened and under circumstances which, in the future, might cause the loss of valuable rights.

It is hoped that the foregoing brief review has indicated a plain tendency for administration of water rights to approach closely the ideal state expressed by the following controlling conditions

(*a*) Complete separation and definition of powers affecting water rights; one body, namely the Courts, to determine the rights; and the administration and distribution of these rights by another agency, the State Engineer.

(*b*) The abolition of all Court Commissioners, their places to be taken by commissioners appointed by the State Engineer, after consultation with the water users.

(*c*) A method of assessments based strictly on water rights with such adjustment as will make them agree approximately with the value of the right in terms of available supply.

DISCUSSION

M. C. Hinderlider,* M. Am. Soc. C. E.—Unquestionably the greatest resource of the arid States of America is their water supplies, and hence the administration problems in this connection are of profound interest to the people of these States. The officials charged with the administration of the Court decrees relating to water supplies, are confronted with many intricate, illusive, and difficult problems.

The laws of most of the arid States relating to the administration of water supplies, lay down broad, comprehensive rules applicable in general to the entire State. Due to the variable character of stream flow, climatic conditions, and the diversity of use of the waters, many conflicts have occurred in the past and others continue to arise to aggravate and confound the officials. Although, in general, these laws have been interpreted by the Supreme Courts of the various States, there are many shadings in such opinions which of necessity leave to an administrative official many intricate questions on which he must pass in administering the duties of his office.

Colorado, next to California, has the largest area of land under irrigation, and has by far the most extensive system of reservoirs and canals. For the most efficient and economical use of her available water supplies, it has been found needful to provide for a comprehensive system of administration which involves the storage of water in reservoirs located both on the channels of natural streams and at points sometimes far removed from such streams; and for a system of exchange between ditches, reservoirs, and the streams.

The Constitution of the State of Colorado provides that priority as to time of use shall constitute the better right as between those taking water for a like purpose. Furthermore, it is stipulated that the right to divert water from any natural stream for a beneficial use, shall never be denied. No limitation whatsoever is placed on any one who appropriates water on any stream, except that of priority of original use and beneficial application.

Colorado has substantially 1 000 storage reservoirs, in which a large proportion of the available water supplies are stored in times of plenty for use in times of shortage. In order that it may be made available to the various canals and ditches, the law provides that water may be released from reservoirs and transmitted down natural streams and taken out again by the water owners, all under the supervision of the State water officials. This one phase alone involves one of the most difficult and illusive problems with which administrative officials are confronted.

The law provides that the same quantity of water released from a reservoir on a stream above, may be diverted by a canal below, minus a certain quantity for loss in transit to be determined by the State Engineer. This loss varies greatly from time to time, depending on the distance between the reservoir and the canal taking the water; the stage of the river and the quantity of water released by the reservoir; the climatic conditions which may prevail at the time such water is being released from the reservoir; the char-

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[*] State Engr. of Colorado, Denver, Colo.

acter of the stream channel, etc. Probably one of the major factors affecting such deliveries is the control of the head-gates of ditches along the stream during such runs of water.

The water officials in Colorado have made many attempts to determine the probable loss of reservoir water in transit under changing conditions, and have found that, due to the variable nature of the many factors involved, it is possible only to ascertain the amount of the losses within certain limits. Such experiments have shown that these losses of water in transit vary from 2 or 3 up to 30% of the water released from a reservoir.

For a proper accounting of all the water supplies in Colorado, the law provides that every reservoir shall be carefully contoured to determine the capacity thereof; and that the reservoirs must be equipped with proper gauge rods, marked in feet and tenths of feet, to indicate the depth of storage therein at any time; that proper measuring flumes or other devices equipped with automatic registers, shall be installed on all streams flowing into the reservoirs and also on all outlets.

The quantity of water in storage at the end of each month in all the major reservoirs is reported to the principal administrative official for that division. While Colorado laws do not divide the calendar year into seasons for direct irrigation and for storage of water, climatic conditions practically effect the same result; that is, when water ceases to be needed for direct application to the land for irrigation, the reservoirs are permitted to store water in order of priority.

When stored water is required by owners along the various streams, it is released from the reservoirs under supervision of the local water officials, measured over automatic registers, and a similar quantity of water delivered to the head of the canal below, minus a certain quantity which is deducted for loss in transit.

Assuming that all head-gates located on the stream between the reservoir and the point of delivery are properly regulated and controlled to insure that other owners do not divert water to which they are not entitled, the only material loss in transit is occasioned by evaporation.

It is true that under certain peculiar conditions loss of water from a stream occurs through subterranean channels which may lead to lower areas adjacent to a stream, but such losses are rather infrequent except for short stretches along a stream.

Some time ago the speaker conceived the idea of attempting to compute evaporative losses from stream channels by first ascertaining the exposed surface area of the channel at different stages of flow, and then applying the proper evaporation coefficient applicable for any particular day. It was soon discovered, however, that the stage of the stream varied radically both day and night, especially in shallow streams with sandy beds and numerous sandbars. This made it impossible to determine with any reasonable degree of accuracy the probable surface area exposed to the air and sun at any time.

As a means of securing more efficient administration of reservoir runs, it was found needful to exercise the most painstaking supervision over the head-gates along the river between the points in transit. To effect such administration it was necessary to require that automatic registers and accurate measuring devices be installed in the ditches. With such equipment supervised by the local water commissioners and their deputies, it has been found that many of the old problems affecting the transmission of reservoir water have been successfully met.

There are eight requisites for a proper and efficient administration of both canal and reservoir decrees and for the transit of reservoir water along a natural stream. These may be listed as follows:

First.—Proper head-gates in all ditches heading on the stream, with facilities for locking them down if necessary.

Second.—Proper measuring devices equipped with automatic registers for determining the quantity of water diverted hourly by the various ditches.

Third.—Numerous gauging stations equipped with automatic registers placed along the stream at strategic locations.

Fourth.—An efficient corps of experienced hydrographers to rate the gauging stations and canals and patrol the river during reservoir runs.

Fifth.—Conscientious water commissioners and deputies to patrol the stream and the head-gates of all canals to see that the water is distributed strictly in accordance with order of priority.

Sixth.—A proper system of telephone communication between the reservoirs, the river stations, all canal head-gates, and the chief administrative official.

Seventh.—A proper system of daily reports between the local water officials and the chief administrative official, showing the quantity of water (both morning and evening) available for distribution; the quantity of water diverted by each head-gate; the quantity of water stored in each reservoir, and in transit down the stream for any ditch.

Eighth.—A proper register of the Court decrees showing the order of priority in which waters may be diverted or stored, and a thorough understanding of these decrees by the administrative officials.

As a further material aid in the administration of reservoir runs and other water decrees, State officials have found that a diffusion of the knowledge of what occurs on the stream each day, or possibly more frequently, by means of a bulletin issued by the chief administrative official or secretary of the ditch organizations along the stream, is most beneficial.

They have found that a complete knowledge of water supply demands and distribution, if made available to the water user, dissipates suspicion, removes grounds for complaint against the administrative official, and provides a permanent record for future reference, which is invaluable.

As time passes and water supplies become more valuable and require a more careful supervision and greater efforts toward conservation, greater efficiency will be demanded, which will inevitably result in improved methods of measurement, control, and accounting.

Mention has been made of gains to the river resulting from the transit of reservoir water along a stream. There can be no actual gain or accretion in the total available water supply as a result of such reservoir runs, but, on the contrary, there must always result some loss of the water in question, due to evaporation and, possibly, to deep percolation away from the stream channel.

It is true, however, that following these reservoir runs, the stream frequently benefits because of the return flow of percolating waters draining out of the sand-bars and land adjacent to the stream channel.

If the percentage of assumed losses chargeable against the reservoir is excessive, the reservoir run suffers. If on the other hand the estimated percentage for loss of water in transit is not sufficiently high, the other rights along the stream suffer.

After releasing water from a reservoir farther up on a stream, and turning an equivalent quantity of water out of the river into the head-gate of the receiving ditch, it frequently happens that some other canal immediately below, which up to that time had been securing its full quota of water, is deprived of all, or some portion, of the water theretofore drawn.

Unless there has been a material change in climatic conditions affecting the natural stream flow, it is obvious that the reservoir run has not been sufficiently penalized, with the result that great injury is done some water user along the river. Since the values of these water supplies are measured daily in dollars and cents, the effect of poor administration has all the results of depriving an owner of actual property with all the resultant dangers of ill-feeling, litigation, property losses, etc.

From the foregoing it may be observed that the intricacies involved in the administration of the water supplies in the Western States and a proper solution of the problems, challenges the earnest and intelligent effort of the administrative official, the best statesmanship of legislative bodies, and the most painstaking thought of the Courts.

R. I. MEEKER,* M. AM. SOC. C. E. (by letter).—Mr. Baldwin has tersely presented his administrative experience dealing with transmission losses incident to runs of several thousand second-feet of reservoir water for stretches of several hundred miles in a large river, in which channel conditions as to gains and losses are variable and administrative problems are complex.

The history of river administration should record an evolution in charges imposed on reservoir water conveyed in river channels. Realization on the part of water users and water officials that there are wide fluctuations in river gains and losses due to stream-bed formations, return flow from irrigation, and other causes, will go a long way toward securing the necessary funds to make engineering studies with which to guide administrative practice.

Continued engineering measurements of river channel flows for gains and losses are necessary in order to fit administration practice to conditions of natural fluctuating river stages, to conform to changing river conditions in irrigated valleys, and to eliminate conjectures and prejudicial assertions frequently made by opposing parties in such matters. Channel losses or gains in a river system may be far different to-day than those of the past or the future. They also vary with the size and compactness of adjacent irrigated areas.

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* Cons. Engr., Denver, Colo.

By statutory enactment in 1879, conveyance of reservoir water in river channels in Colorado was made subject to deduction for evaporation and seepage losses. Subsequent legislative acts in 1897 imposed a "reasonable deduction for seepage and evaporation" on waters transferred from one stream to another, or on exchanges of reservoir and ditch water. The State Engineer was designated as the agency to make determinations of losses applicable to such waters.

In framing legislation concerning conveyance losses of reservoir water, inter-water-shed diversions, and exchanges of water, the underlying principle has been "the non-injury to other water rights upon a stream system." In administrative practice, reasonable doubt, therefore, has usually been resolved against the reservoir water.

Notwithstanding engineering information on river channel losses, the question of penalty losses applicable to reservoir waters transferred in river channels is fertile ground for water disputes, especially in dry years when the effect of water shortage is felt. The writer's experience on losses applicable to reservoir water in transit in river channels convinces him that frequently grounds for complaint by direct-flow users are justified. Actual losses sustained by prior users sometimes occur from weak administration of intervening diversions, in river sections between the reservoir and the diversion works, rather than from actual channel losses by seepage. Evaporation losses of water in transit usually are negligible when compared to percolation or administrative losses. Seepage and administrative losses are frequently confused.

TABLE 2.—TRANSMISSION LOSSES IMPOSED ON RESERVOIR WATER.

Reservoir.	Stream.	River channel distance, in miles.	Character of river bed.	Deduction, percentage.	Basis of determination.
Twin Lakes....	Arkansas......	160	Chiefly through mountains.	10	By stipulation; engineering studies under way.
Rio Grande....	Rio Grande....	80	Through mountains.	8	By stipulation.
Antero.........	South Platte ..	100	Through mountains.	12.5	Stipulation modified by engineering studies.
Cheesman	South Platte ..	25	Through mountains.	2.5	By stipulation and stream-flow records.
Cheesman	South Platte ..	55	Through mountains and plains.	5	By stipulation and stream-flow records.

In mountainous country, channel percolation losses are usually quite small. In plains and valley sections of rivers, both seepage and administrative losses may be heavy. River channels through lava (basalt formations) are likely to be erratic as to losses or gains, and should be evaluated by actual measurements. As river channel losses are seldom directly comparable, the application of percentage deductions for one river section to other river sections, or from one river channel to another, is dangerous.

On account of lack of funds for engineering studies in advance of reservoir runs, transmission losses applicable to "reservoir runs" generally have been set by stipulation of the interested parties and water officials, and, later, have been checked by engineering measurements and modified in accordance therewith. The time has come for engineers, water users, and administrative heads to insist on funds for investigating these matters. The loose haphazard methods of the past concerning penalty losses applicable to reservoir and interstream waters should be relegated to oblivion. Problems of dwindling water supplies, increasing use of stored waters, and changing river conditions due to irrigation, power, and industrial uses, demand engineering treatment.

Table 2 outlines roughly the transmission losses charged against reservoir water in Colorado. The data therein contained are taken from records of the State Engineer's Office at Denver.

LYNN CRANDALL,* M. AM. SOC. C. E. (by letter).—The construction and operation of storage reservoirs on natural stream channels in the irrigated areas of the Western United States have resulted in more or less contention between the owners of the stored water and the owners of earlier natural flow rights. The disputes thus occasioned generally pertain: First, to the segregation of stored water and the natural flow as discharged from the reservoir; and, second, to the proper transmission loss to be charged to stored water while in transit in the stream channel between the reservoir and the place of diversion from the stream.

The practice of determining the daily amount of storage released, from reservoir drop and capacity tables, as used at Jackson Lake and Henrys Lake Reservoirs and elsewhere, is probably the simplest method to apply. In all cases it is useful as a check on other methods, but its application causes all errors and fluctuations from erroneous capacity tables, evaporation losses, return ground storage, etc., to be thrown into the determined daily natural flow. This, in turn, causes fluctuations in the determined natural flow from day to day that would not exist except for the construction and operation of the reservoir and the use of this method of segregating the stored water from the natural flow. While such fluctuations may balance throughout the season, the natural flow rights are often of varying dates of priority, and during periods when the stream flow is not sufficient to fill all the natural flow rights, this method may deprive certain of such rights of water, to the advantage (at some other time during the season) of other natural flow rights of different dates of priority. For example, in various reservoirs with which the writer is familiar the water held as ground storage amounts to from 3 to 10% of the open reservoir storage as determined from the contour surveys, and during the course of the season this may compensate fully for evaporation losses. However, such losses are generally greatest when the reservoir is full, while the return from reservoir ground storage usually reaches a maximum later in the season when the reservoir is being lowered rapidly. Under this method of operation, then, the evaporation loss may cause a certain natural flow right of late priority to be cut off earlier than it should be, while the ground storage gain accrues to an earlier right still being filled later in the season. On reservoirs with numerous fluctuating sources of supply, however, this method is about the only practical one, but in such cases its use should be supplemented by special investigations like those made at Jackson Lake. Records of all inflow, outflow, evaporation, ground-water fluctuations, etc., should be secured so as to afford a basis of evaluating the effect of operation on any particular rights involved, as well as a means for calculating a reservoir capacity table, based on inflow and outflow records, independent of that determined from contour surveys.

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* Water Commr., Big Lost River, Idaho Falls, Idaho.

Where there is no natural gain to the stream in the reservoir site, or where such gain, if any exists, comes from springs that are fairly uniform in flow, it will usually be found more satisfactory, fairer to every one concerned, and less productive of dispute, to determine the reservoir inflow each day from a gauging station, or from stations situated above the back-water of the reservoir. Then add to or subtract from such inflow an agreed or determined flow for natural gain or loss in the original stream channel through the reservoir, thus securing the natural flow passing the gauging station below the reservoir, any surplus at that point, over such determined natural flow, being deemed released stored water. This method is prescribed in the Court decree governing the distribution of the waters from Big Lost River, Idaho. An allowance of 34 sec-ft. is made for natural stream gain in the Mackay Reservoir, which is based on records compiled during periods when the reservoir was empty. A similar method is followed in the operation of the Magic Reservoir on Big Wood River, Idaho. On Big Lost River, it has been found necessary to locate the measuring station at least 2 miles below the dam, in order to get far enough down stream to intercept all the ground-water outflow from the reservoir, more than 100 sec-ft. of which flows around or under the dam itself when the reservoir is full. Even in the case of reservoirs with a large inflow from springs (provided records are available prior to the reservoir construction or during periods when it was empty), it may be more satisfactory and equitable to determine or agree on a figure, to represent such natural gain through the reservoir site. If the records justify it, a slightly different figure should be used for different times during the season, rather than rely upon some method which might result in unwarranted fluctuations being caused in the calculated natural flow.

Having segregated, by some means or other, stored water from natural flow as it leaves the reservoir, the question of proper transmission loss to charge stored water as it flows in the natural channel, mingled with natural flow, to the point of diversion from the stream, still remains as a source of contention. In this case, as in the case of segregation of water at the reservoir outlet, it would seem fair to start with the basic assumption that the natural flow rights are entitled to the quantity of water that would be available at their points of diversion if the reservoir had never been built or if stored water had never been carried in the stream, and that any additional amount available over and above such quantity belongs to the owners of the stored water.

The loss occasioned by running stored water in natural stream channels will vary widely, depending on whether the stream naturally loses or gains water in the section considered. The head-water areas of most Western streams are regions of fairly heavy precipitation, where the stream gains from tributary ground-water inflow. As the stream emerges from the mountain canyons into the lower valley, it often flows for some distance over a delta or cone where the adjacent ground-water level is lower than the water level in the stream, and where the natural channel loses water at all stages. Farther down stream the ground-water may again rise higher than the water level in the stream, resulting in a ground-water inflow, and it may then gain to the stream in such sections. On numerous Western streams such alternate sections of gain and loss occur for many miles.

In sections where the stream loses water due to the adjacent water-table being lower than the level in the stream, the stored water should manifestly be charged with the additional loss created by carrying it, as compared with what the natural loss would be without including the stored water. The practice of determining percentage losses in the channel with both stored water and natural flow in the stream, while supplying information of interest, should not be relied on entirely to determine proper losses chargeable to stored water. A comparison of losses with natural flow alone, and when carrying the additional stored water should also be made. Studies of the ground-water flow through the valley must necessarily be made to determine whether the water lost in the "loss" sections of the stream returns farther down stream as gain and, if so, when and where such return takes place. Stored water is properly entitled to credit for whatever portion of the "lost" stored water that returns to the stream above diversion points during the period of full use of the stream flow each year. In some of the larger river valleys of the West, this "lost" water requires several years to complete its underground travels and finally emerges as ground-water or spring inflow into the stream. Thus, the continual carrying of stored water for year after year may result in that portion of such stored water that is lost in the "loss" sections, finally re-appearing as a permanent increase in the gain to the stream in the "gain" sections.

In sections of the stream where the water-table is higher than the water level in the stream channel, the stored water sustains little actual loss, but it has the effect, when first turned in, of damming back part of the natural ground-water inflow until the ground-water slopes adjacent to the stream re-adjust themselves to the higher water level in the stream occasioned by the stored water. The effect of running stored water in such sections of natural gain from ground-water inflow to the stream can best be studied by noting the reduced gain for a short period, varying under different conditions from a few days to several weeks, when stored water is first turned into the channel. This temporary bank storage returns rapidly to the stream when the stored-water run ceases and the stream drops to its natural level. In the delivery of stored water on Big Lost River, the matter is adjusted by allowing the natural flow rights to use some of the stored water during the first week that it is turned into the stream, the same quantity being repaid to the stored-water owners later in the season from the increased gain that occurs for a brief period when the stored-water run ceases.

As pointed out by Mr. Baldwin, the matter of the equitable segregation of stored water and natural flow is a complicated one in many cases. It is not subject to exact determination on account of differences in climatological factors, ground-water levels, stream channel conditions, etc., from year to year and at different times during the same year. By securing records covering a number of years on inflow, diversions, and losses along the stream, together with studies of the ground-water movements, however, a basis may be developed for an intelligent study of the question, and a fair approximation may be thus reached for the determination of the natural flow and stored water, as each is delivered to its respective owners.

THOMAS R. NEWELL,* ASSOC. M. AM. SOC. C. E. (by letter).—An outline of the methods used in transmitting and delivering reservoir water in Water District No. 36, Idaho, has been presented by Mr. Baldwin. It is mentioned that transmission is effected through the Snake River channel from both the Jackson Lake and American Falls Reservoirs to the river head-gates of the numerous owners of storage rights. Thus, the channel of the main river carries an increment of reservoir in addition to the current run-off of the water-shed during a large part of the irrigation season. Natural water distribution must be made simultaneously with storage delivery despite the complications introduced by mingling this current run-off with these continuous streams of transit storage. In some sections of the river the reservoir water increment is even greater than the natural flow during part of the season. The relative proportions vary for years of differing snow supplies and irrigation demands.

Transmission loss schedules and reservoir outlet divisions always must meet the tests of changing conditions and even of changing ownerships. The small percentage of ultimate wastage (flow past Milner Dam during regulation period) is a real index of the excellence and efficiency of the control system of river operation as a whole. The fact that these major reservoirs have been operated ever since their construction in conjunction with natural water distribution from the same water-shed, through identical delivery channels and without the authority of a Court Commissioner, speaks well for the efforts of the administrative official and perhaps also for the tolerance of district water users.

The compromise agreements upon which division is made between natural and reservoir waters when flowing commingled are of particular interest to the writer. In Water District No. 36 these agreements consist of brief digests of methods of division at reservoir outlets and schedules of percentage transmission losses to be applied through successive river sections to transit storage. As they are adopted annually in slightly revised form, these compromise schedules should carry greater prestige from year to year, and as time goes on, should become more nearly representative of true conditions. It is readily seen that variations in these schedules directly affect the quantity of reservoir delivery. In like manner, changes affect the total supply of natural water in the opposite direction. Even now basic changes are stubbornly contested and are almost impossible of adoption without recourse to hydrological investigations of a co-operative nature. Objections of merit are usually met by secondary arrangements or the granting of special privileges to avoid upsetting standard compromise schedules.

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In this connection it is interesting to note the exact effect of a change in a river operation agreement from the viewpoint of a holder of a natural water decree and a user of reservoir water. Assume, for example, that a transmission loss schedule is increased 10 per cent. Each storage owner's holdings are decreased 10 per cent. The only escape he has from the penalty is that his storage right may be a mere insurance and seldom withdrawn. This same increase in applied transmission loss increases the total supply of natural water, but not *pro rata* to each holder of a decreed right. In Idaho, a decreed right specifies a quantity (if beneficially used) and, in all cases, a priority date. As river supplies decrease below the quantity necessary to fulfill all decrees in use, the river water master refers to his chronological list of priority dates and declares that right with the latest date either partly or completely invalid and denies use thereunder. This process of eliminating one user after another is continued until the river stage reaches the summer low. Then, as supplies increase, each decree is restored in reverse order.

Pioneer rights to the extent of the minimum summer flow remain untouched and, in reality, have no interest in the changed transmission loss. The younger the right the longer is the period of denied use which must be suffered. To each invalidated right an increased transmission loss means an extended time of validity at either end of its period of denied use. The length of the extension is defined by the amount of gain to natural supplies due to the changed schedule and the rate of river decline or increase. It may not encompass the whole decree and it may not help for more than a single day; but for several years the storage delivery rate from the Jackson Lake Reservoir has been quite large during the months of July and August. Likewise, the summer minimum of Snake River flow usually remains fairly constant for several weeks in the height of the irrigation season. Therefore, it is possible for a single large decree, properly placed along the chronological scale, to garner a lion's share of the total increase in natural supplies resulting from a changed transmission loss.

To carry the illustration still further, it is true that many of these young rights have purchased supplemental storage, so that their gains may be partly offset by losses to their supplemental supplies. Careful study of these effects serves to accent the fallacy of the popular belief that any general method of division between the two classes which balances for the season is fair to the individual decree holder. It may also present to the politically minded an interesting angle of the "ins and outs" of stipulative agreements.

Compromise river and reservoir agreements should tend toward workable methods of division which should be fair to each party throughout each season and which should keep pace with current events. With the initial draft from the new American Falls Reservoir comes a series of changed ownerships, new river complications, and notable variations in river regimen extending from Jackson Lake all the way to Milner. A reservoir outlet division, operated on the contour method, in which evaporation losses and natural storage credits are balanced against bank storage return, usually contemplates a full use of the reservoir and cannot be expected to serve a hold-over storage unit. Henrys Lake Reservoir is notably in the latter class and on account of recent changes in ownership at Jackson Lake, this great storage unit is tending in that direction.

These same factors operate toward less regular increments of reservoir water being carried through the main river channel. Flat percentage rates of transmission loss and flat time interval allowances for any and all river stages and rates of delivery will not so fairly comprehend the new situation. Lag credits help to even the charges, but should be applied in direct proportion to increases in stage and should not be permitted to be used in any but a regular mechanical manner. The space privilege, the temporary transfer, and many permanent transfers might each serve an honest purpose did they not so often go hand in hand with appropriation and even Court decree beyond the bounds of beneficial use. The current dearth of water supply in Water District No. 36, due to the many new storage purchases (made possible by the construction of the American Falls Reservoir, and the ample natural run-off of 1927 and 1928), has eased the river operation situation temporarily. The advent of a season of shortage will not soon bring general crop losses on account of the presence in the district reservoirs of much "water insurance" which may be rented; but with the reclamation of additional lands, now contemplated, supplies will eventually be depleted to the point where river and reservoir problems will become more important.

Only a part of the transmission and division agreements now in force have support in actual seepage investigations. Of these investigations, many have been continued during one season only, while some are based only on miscellaneous field runs. Continuity through different types of seasons—the fundamental of every thorough hydrological study—has been denied in most cases. Few, if any of the channel studies made in the past, show the application of the modern methods of refining river-station differences in use in California and some even ignore surface tributary inflow as an independent variable. The Kootenai River study now in progress (1928) on the International Boundary has been planned with a view to obtaining a degree of refinement far beyond that of any lake or reservoir segregation yet made in this district. The criticism often heard, to the effect that the segregation of the many factors involved can never be exact, is true despite the use of the most modern methods economically feasible.

However, real values do result from orderly presentation of measurable factors properly qualified as to exactitude. Just divisions require careful observation of seasonal trends of disputed factors. Developed waters resulting from drainage recoveries, return flows from new irrigation, artificial ground storage, and reservoir bank storage, should each be observed currently. Changed plans for reservoir drafts due to changing ownerships should not be overlooked. All in all, too much should not be expected of old and insufficient investigations, and every effort should be made to make the new ones more thorough and continuous. The effort to centralize and co-ordinate all administrative activities, within the district, has built up a continuing and an adequate organization which has effected improved operation service. The basic schedules underlying operation schedules are entitled to equal consideration and support.

To the student of river operation, Mr. Baldwin's paper is particularly interesting, in that it is representative of ten years' experience in transmitting reservoir waters at rates of much greater magnitude than usual. The same rules which apply to storage deliveries at the rate of a few miner's inches for a few miles are entirely inadequate in transmitting reservoir water by the thousands of second-feet for hundreds of miles to the respective owners. It is to be regretted that more detail has not been included for a careful description of the very effective system of control which has resulted in uninterrupted service and, at the same time, a negligible ultimate waste. The attainment of these two ends is the mark of good regulation and deserves the careful consideration of every person interested in orderly distribution on a large scale.

E. B. **Debler**,* M. Am. Soc. C. E.—The State of California adopted riparian rights as the basis of its water rights many years ago, and as no affirmative action on the part of the grantee was required, such rights became vested as to all riparian lands, with the action of the State Legislature.

Fundamentally, riparian rights imply non-consumptive uses only, but in the main the Courts have upheld any uses which did not deprive other riparian owners of their proportionate share of such use.

Recent attempts to impose limitations on the use of these riparian rights, even if only in support of a more efficient use of the State's water resources, either by legislative enactment or by regulations of an administrative arm of the State Government, are likely to prove fruitless as they would appear to be in the nature of confiscation of a vested property right which the Courts will not sanction. In view of this situation, plans for the conservation of the water resources of the State must be placed on a sounder legal basis.

The situation with regard to conflict of riparian and appropriation rights is not limited to California. Both types of rights exist in a number of Western States. Nebraska has adopted riparian rights for its humid section and appropriation rights for its arid section. Land grants by the Spanish Crown in the Southern Border States often carried water rights, sometimes distinctly "appropriation" in character, at other times, "riparian." The outcome of California's attempts to curb riparian rights will be watched with interest, as it is likely to be followed in after years by similar attempts in other States when water becomes of greater value.

G. **Clyde Baldwin**,† M. Am. Soc. C. E. (by letter).—Discussion of the writer's paper has very properly not been limited to the problems incident to the transmission and delivery of water from Snake River reservoirs. Mr. Hinderlider has presented an excellent summary of the methods used in the State of Colorado and has listed eight so-called requisites for equitable and efficient water administration which, with possibly slight modification, should be generally applicable.

The extent to which transmission losses are increased by "adverse-user" diversions, together with the accessibility of their head-gates, obviously will determine the degree of regulation which should be enforced. For example, it would probably be unwise to attempt to prevent the use of water by isolated canyon ranches unless the resulting benefits to be derived by the owners of older rights would be sufficient to justify the expense of the regulation.

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Mr. Meeker emphasizes the desirability of continued, comprehensive, engineering investigations and reports as an aid to better administration, and this is also stressed by Mr. Newell. Both, however, recognize the variable, complex character of the different factors on which the stream flow at any given point depends, and they understand that exact solutions of river or reservoir loss problems can never be attained.

Engineers usually are anxious to secure an increasing amount of data on such subjects in order to minimize the always recognizable error in the final application.

From the practical viewpoint of the ordinary water user, on the other hand, extreme accuracy seems less essential, especially if the engineering investigations must be done at his expense. Furthermore, in such extremely complicated problems as those on Snake River, the same basic facts may be used sometimes in good faith by different engineers to derive somewhat divergent conclusions. The water users, therefore, as a group, are rather slow to adopt radical changes in methods of operation and prefer to follow the usage of preceding years, with perhaps slight modifications based on new facts, rather than to sanction expenditures for engineering investigations which may result in controversy.

As mentioned by Mr. Crandall, segregation of stored water and normal stream flow at the outlets of reservoirs and the schedule of losses to be charged the former while in transit in the stream channel between the reservoirs and points of diversion are usually the two principal sources of contention in river administration.

The desirability of having these points settled by Court order or decree has long been recognized by those most interested in the storage and delivery of water on Snake River, and much has been accomplished through special studies and investigations with that end in view. At present (January, 1929), reports covering ground-water conditions along Snake River and the 1928 gains and losses in the American Falls Reservoir Basin are in process of preparation.

The net result of more than ten years of river operation has been the plan described in the paper which, as modified from time to time to meet new conditions or to be more consistent with added facts as they became available, has now, through precedent, attained a position almost comparable to, and may eventually be converted into, a Court order through stipulation by the interested parties.

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Paper No. 1730

RETURN WATER AND DRAINAGE RECOVERY FROM IRRIGATION*

A SYMPOSIUM

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With Discussion by Messrs. R. I. Meeker and Fred H. Tibbetts.

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* Presented at the meeting of the Irrigation Division at Denver, Colo., July 14, 1927.

RETURN WATER, NORTH PLATTE RIVER, NEBRASKA

By R. H. Willis,* Esq.

Synopsis

The subject of return water is an interesting problem in the North Platte Basin. To determine, with a reasonable degree of accuracy, the quantity of return water in any irrigated area it is necessary that full co-operation be maintained between the State Department and the managers of those projects diverting water from the stream. Experience shows that it takes several years to obtain the necessary co-operation from project managers. In Nebraska, it has not yet been fully attained.

Quantities

The quantity of water returning to the North Platte River, after being diverted for irrigation, is amazing to those who have not observed the habits of this stream. The mean annual flow since about 1900, at North Platte, Nebr., is 2 294 000 acre-ft. This mean is not likely to change materially by having more years of record. Approximately, 1 200 000 acre-ft. have been diverted from the stream during each of the seasons of 1925, 1926, and 1927, between Whalen, Wyo., and Bridgeport, Nebr., and were conducted to the farming lands in the basin. About 65% of this water came back to the stream. This water is known as "return water", "return flow", "seepage", and "unused water."

In 1909, there were probably fewer than 50 000 acres under irrigation in the valley west of Bridgeport. Not until 1912 did the visible return flow become perceptible. After 1909 much additional land was put under irrigation between Whalen and Bridgeport. By 1919 the total had reached 233 000 acres. At present, (1927), the irrigated area in the valley comprises approximately 350 000 acres.

Elements Contributing to Return Flow

Since 1910 there have been two main factors affecting the habit of the river, namely, the storage of flood water in the Pathfinder Reservoir, and the increased diversions from the stream. The reservoir serves as a direct retard of the flow of the stream from the non-irrigation to the irrigation season. The effects which flood storage and return flow have on the lower section of the North Platte River are shown graphically in Fig. 1. The mean monthly flow before and after the completion of the Pathfinder Reservoir is shown for each month of the year. It will be noted that since the construction of the reservoir the mean flow for May and June has been approximately 50% less than the mean prior to that time. For July, the mean flow has been

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30% less; for August, 95% more; for September, about 225% more; and for October, 215% more than the mean flow prior to the beginning of storage in the Pathfinder Reservoir. This increased flow during the months cited is the result of the application of water to the irrigated area between Whalen and North Platte.

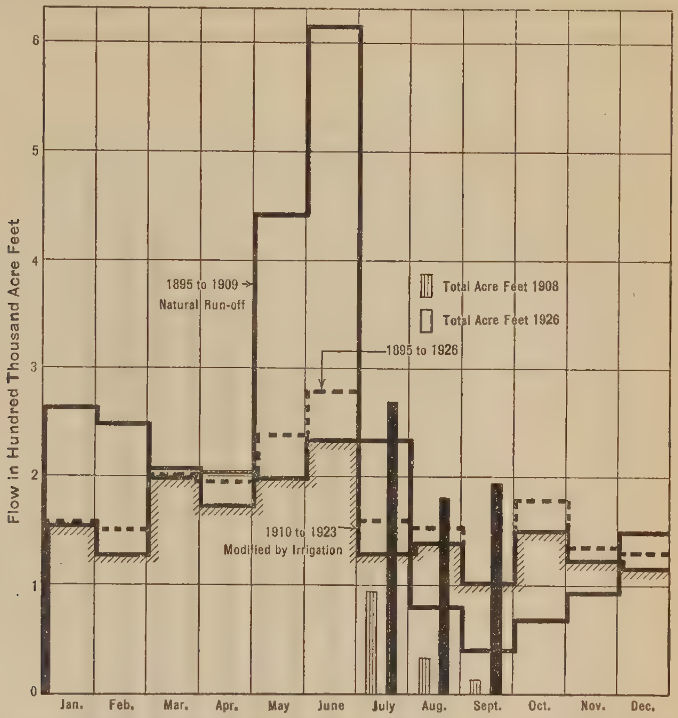

FIG. 1.—MEAN MONTHLY DISCHARGE OF NORTH PLATTE RIVER, AT NORTH PLATTE, NEBR.

The increased diversion of irrigation water has been the major contribution to the return flow. It is generally admitted that approximately 1 sec-ft. of water is consumed for every 3 sec-ft. diverted. The 2 sec-ft. become return flow after serving as a vehicle for carrying the 1 sec-ft. consumed. Part of the unused water is recovered by drainage canals. Although there are many miles of these canals in use in the North Platte Valley, many more miles must be constructed before return water is efficiently brought back to the stream. It must be kept in mind that some of the unused water returns to the stream through channels other than the drainage ditches. Not all the unused water is returned through drainage canals. Probably the major portion is returned through deep percolation designated as "invisible return flow."

RECORDS OF FLOW, RAINFALL, AND TEMPERATURE

Visible return flow, as herein used, includes some waste water. Nearly all projects use drainage canals occasionally to conduct excess water back to the stream, and it has not been practicable to separate the waste water from the seepage. There is also surface water from irrigated fields bordering along the drainage canals that must be taken as visible return flow. The visible return flow of the North Platte River between the Wyoming-Nebraska Line and Bridgeport amounted to 630 000 acre-ft. for 1926. Fig. 2 shows the yearly quantity of visible return flow each year from 1912. The three rather pronounced jumps—1916, 1919, and 1924—are accounted for probably by preceding wet years.

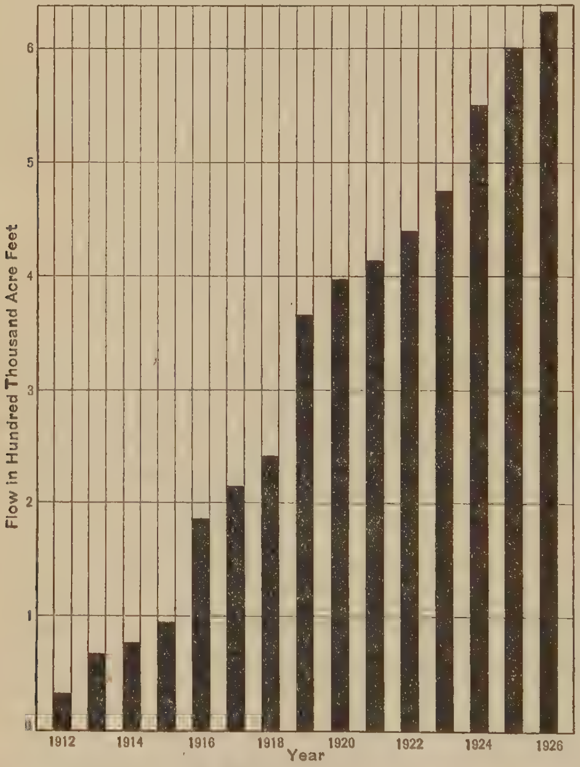

FIG. 2.—VISIBLE RETURN FLOW BETWEEN WYOMING-NEBRASKA STATE LINE AND BRIDGEPORT, NEBR.

The records of precipitation between Whalen and Bridgeport furnish the following information: During the irrigation season (May to September, inclusive) of 1915, the rainfall was 8.5 in. and the year's rainfall was 11.6 in. above normal; the record for the 1918 irrigation season showed 5.3 in. and the annual precipitation was 7.0 in. above normal; and for the season of 1923 the recorded rainfall was 4.6 in. and the annual quantity was 4.4 in. above normal. These are the greatest precipitations recorded since 1912, and the three wet years preceded the years that show the marked gain. The years 1916, 1919, and 1924 show the lowest precipitation since 1912, with the exception of 1914, which was 6.0 in. below normal.

The temperature during the irrigation seasons was below normal in 1916 and 1924, and considerably above normal in 1919. High temperature and hot winds diminish the visible flow. In Western Nebraska water is diverted for irrigation between May 1 and October 1. The diversions are greatest in July and August, and the return flow, in September and October, the lowest being about the last of April. The quantity of visible return flow available for use during the irrigation season of five months is approximately 48% of the annual visible return flow.

VARIATIONS IN RETURN FLOW

At Whalen, the United States Bureau of Reclamation operates and maintains a concrete diversion dam. Immediately above the dam two large canals, one on each side of the river, divert from the river during the irrigation seasons a quantity of water varying from 2 000 to 3 500 sec-ft. Special efforts were made to procure accurate data on the return flow between Whalen and Bridgeport during the years 1925, 1926, and 1927. There was a gradual increase in return flow from station to station down the river. For example, in August, 1925, it was found that there was no return flow at Whalen. The flow gradually increased at successive stations down the river until at Bridgeport, a distance of 93 miles, the mean daily return flow was 1 869 sec-ft., of which 57% was visible return flow. The remainder was invisible coming into the river from deep percolation and probably not all should be classed as return water. Rainfall in the basin contributes considerably to the invisible flow and causes it to vary. It is impossible to determine what part is rainfall.

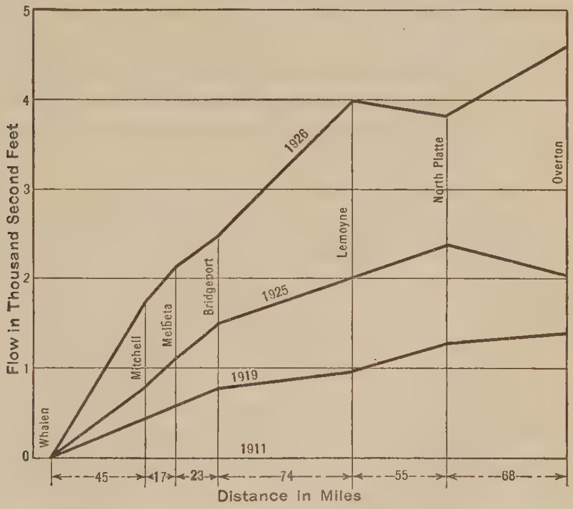

FIG. 3.—TOTAL ACCUMULATIVE RETURN FLOW, FIVE MONTHS.

The quantity of water returning in a comparatively short time is remarkable. Fig. 3 shows graphically the marked increase. This diagram includes both visible and invisible flow. There is a very great increase in 1926, and the records for 1927 confirm the 1926 curve. The precipitations for 1925, 1926, and 1927 were practically normal. The temperature was above normal in 1925 and 1926 and considerably below normal in 1927. The diversions for 1925, 1926, and 1927, between Whalen and Bridgeport, were 1 229 000, 1 935 000, and 1 171 000 acre-ft., respectively. The visible return flow shows in Fig. 2, a uniform increase for these three years. The astonishing increase for 1926, as shown in Fig. 3, is the invisible flow, which comprises return water through deep percolation from the irrigated area and precipitation through deep percolation from the entire area of the drainage basin. Years of large precipitation and low temperatures are favorable to greater invisible flow.

No storage water was available for use in the valley until after the completion of the Pathfinder Reservoir in 1910. Since that year storage water has been used to supplement the direct flow. During the 1927 season, storage water was used on approximately 184 000 acres between July 1 and September 30. The direct flow was used on the remaining irrigated area between May 1 and September 30. The direct flow comprises the natural head-waters of the Platte Rivers, together with the return water from the irrigated area of the basin. The return water has been sufficient for all diversions made east of Mitchell, Nebr., since 1924, or longer. The return flow between Bridgeport and Overton, Nebr., comprises only a small part of the total flow of the stream for the reason that the irrigated area between the two stations is comparatively small. During the seasons of 1925, 1926, and 1927, about 270 000 acre-ft. were diverted for irrigation purposes in the territory east of Bridgeport.

DRAINAGE RECOVERY FROM IRRIGATION

By D. W. Murphy,* M. Am. Soc. C. E.

Synopsis

In the practice of irrigation a part of the water applied sinks below the zone of plant growth and is wasted. It is possible, in many instances, to recover a portion of this underground waste through drainage. The quantity that can be recovered depends largely on the character of the underground formation.

The feasibility of recovering irrigation wastes through drainage depends on the cost of draining the water out of the subsoil and its value at the point where it can be collected and discharged. On areas where drainage is necessary to protect soils from becoming water-logged and alkaline, the value, if any, of the water recovered is a net gain.

In the Salt River Valley in Arizona the water pumped to drain and protect the soils is a valuable asset: First, for supplementing the gravity supply during dry years; and, second, for sale for use on adjacent lands.

What is Meant by Drainage Recovery

The term, "drainage recovery", like many others used in irrigation practice, is susceptible of various interpretations. In this paper it is applied to waters used in irrigation, which are not taken up by plants or evaporation and so can be removed from the soils through drainage and made available for further use. It is, so to speak, the underground waste from irrigation which may be recovered from the soils before it finds its way to some natural watercourse. Surface waste from irrigation, that is, waters which merely flow over the lands and off them as surface flow, are not included.

Return flow from irrigation differs materially from drainage recovery. It is water that finds its way through the soils downward and by lateral percolation to natural watercourses. On the other hand drainage recovery may, and ordinarily does, reduce return flow through cutting off all or a part of the water that would otherwise find its way to natural watercourses. It tends also to prevent evaporation from the surface by maintaining the water-table at a lower elevation and thus reducing the quantity of water brought to the surface through capillary action.

Consumptive Use of Water

The total quantity of water applied in irrigation may be stated in terms of consumptive use by plants, of surface evaporation, and of waste, both surface and underground. Drainage recovery is dependent, it may be said, primarily

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on the last factor, underground waste. The amount of underground waste varies greatly for different conditions. The adequacy of a water supply, its cost, the method of application, and the character of soils, all may influence it.

From the few determinations that have been made on the consumptive use by plants, it appears that a large part of the water applied in irrigation is lost either through evaporation or waste. In studies made over large areas in Southern California, the conclusion was reached that 1.82 acre-ft. per acre per year represented the average consumptive use by plants. From this, it appears that with ordinary irrigation a large part of the water applied is lost or wasted, so far as the area to which it is applied is concerned. It indicates also the probability of considerable underground waste and the possibility of large quantities being recovered through drainage.

Water That May Be Recovered

The quantity of irrigation water that can be recovered can never exceed that of underground waste, and it may be but a small, even a negligible, part of such waste. The factors that control the amount of recovery, assuming that the underground waste is a material quantity, generally include soils and the character of the underground formation. It is possible to conceive of an area lying in a closed rock basin out of which no waste from irrigation can escape. In such a case it is possible, through drainage, by pumps and wells, to recover all, or practically all, the irrigation waste from the given area.

The same is also true, but in a somewhat lesser degree, in many valleys with relatively narrow outlets. Practically all the waste into such valleys can be pumped out by a proper and judicious placing of drainage wells. Such areas exist at many places in the Valley of the Rio Grande, and also in those of other Western rivers. The conditions necessary for this kind of action are: First, a closed or nearly closed valley for holding the irrigation water; and, second, a valley fill sufficiently pervious to permit relatively free passage of water through it and to serve as a medium in which the waste waters may be collected. These represent extreme conditions favorable for drainage recovery.

It is equally possible also to conceive of an area not bounded by any rock or other impervious materials, from which underground waters can escape with relative freedom. Where this is the case the irrigation waste flows away more freely, and little or none of it can be collected by drainage works. Such areas are sometimes found on sandy or gravelly mesas adjacent to streams, and in valleys which widen rather than contract toward their lower ends.

Another condition which makes difficult the collection of drainage water is a shallow, comparatively pervious top stratum underlaid by impervious materials. The drainage of such a formation ordinarily must be accomplished by cutting into the impervious materials and collecting the waste water in such excavations by the slow process of flow or percolation over the top of the impervious stratum. Such a condition permits of little storage capacity in the soils. Furthermore, it is difficult to collect the drainage water in large quantities, except through a long system of drains. In such cases the drainage discharge may be at the extreme lower portion of the irrigable area.

Feasibility of Recovery by Drainage

There are several practical and economic questions to be considered in connection with drainage recovery of irrigation water. Among these may be mentioned the following:

(1) The quantity of water that can be recovered.
(2) The quality of water, that is, its fitness for use in irrigation.
(3) The cost of recovery.
(4) The location at which water can be delivered.

The quantity of water that can be recovered may vary from nothing to possibly as high as 20 or 25% of that applied in irrigation. It may even exceed this percentage on limited areas. Large underground wastes in irrigation are most commonly experienced during the early stages of a development. This is due in large part to the limited experience of settlers in irrigation and also to the more abundant water supply available in the first stages for development. Experience together with the greater demand and consequent increased value of water, tends gradually to reduce waste in irrigation. For this reason a water supply which depends wholly, or in large part, on drainage recovery may be reduced in future years, due to more economic and better use of water in irrigation.

It is clear that irrigation waste is due, in large measure, to man's inability to deliver water to land in the most efficient manner. Further, it is impossible to predict with any degree of certainty how much water can be recovered from a given area in advance of actual irrigation and drainage of that area. Information concerning the quantity recovered from one area is of little value for another, due to the many variable factors in each.

Quality of Drainage Water

The character of drainage water must be given consideration in every case where it is to be used for irrigation purposes. This feature is difficult, due to the fact that there is no fixed standard by which the quality can be readily compared. There was a time when soil chemists were willing to state with some degree of positiveness that water containing more than a certain amount of harmful alkali salts was unfit for use in irrigation. Many contradictory experiences have somewhat modified these ideas, so that now most soil chemists and engineers are willing to admit that water containing slightly more than this specified amount may not be unfit for use and that if it contains less than the specified amount, it may not be good.

Cost of Drainage and Value of Water Recovered

The cost of draining irrigable areas ordinarily is an obligation that must be met for the protection of lands, whether or not the water drained off can be utilized. Up to a certain point the cost of drainage works cannot be charged against the supply attained through drainage. It frequently, happens however, that the expense of conveying a drainage supply to a locality where it can be utilized is high, and consideration must be given to it. This leads at once to the fourth consideration as to where the recovered drainage water can be delivered.

In many areas of extreme aridity where the water supply is the controlling factor in determining the extent of agricultural and other development, attention should, and is being, given to drainage recovery to supplement other supplies that are well nigh exhausted. In such cases the value of water is generally sufficient to justify any reasonable expense that must be incurred in order to obtain a supply.

In areas where there is more water than is required to irrigate the available supply of land, economic questions are of first importance in drainage recovery. In any event each individual case must be given independent study and consideration. There is not sufficient experience to permit of such generalities as that all drainage water should be utilized. Neither can it be said that it is not feasible to recover drainage water, since it has been shown through actual experience that in some localities it is economically and otherwise feasible.

It is evident, in the truly arid regions, that agricultural and other developments will require all the available water supply within a few years. It is necessary in such areas to give consideration to conserving the entire supply. This will require a curtailment of irrigation water or a recovery of such wastes through some form of drainage. It is reasonable in this connection to assume that no single method can or will be used in such conservation, but that waste from irrigation will be reduced to a minimum through better methods of use, and that unavoidable waste will be recovered as far as practicable.

Example of Drainage Recovery

The most noted instance of recovery of water through drainage that has come within the writer's experience is in the Salt River Valley in Arizona. The irrigable area within the project is about 200 000 acres. After irrigation had been in progress for a number of years the water-table rose, over a large part of the area, sufficient to threaten the irrigability of the soils; and over a considerable area, sufficient to render them unfit for cultivation, due to water-logging and the accumulation of harmful alkali salts on or near the surface.

The plan considered most feasible for draining the valley was by means of wells sunk to depths varying generally from 150 to 300 ft. The formation of the valley is such that pumping from deep wells has a direct effect on the elevation of the water-table; that is, the water-bearing materials of the upper and lower portions of the valley fill are sufficiently connected to cause a general lowering of the ground-water by pumping from deep wells.

In deciding on the plan of using wells for drainage, it was the intention that the water pumped from the soils would be used, in so far as possible, to augment the natural irrigation supply for the project which comes from Roosevelt Reservoir. With this end in view, wells were located, consistent with drainage requirements, so that water could be pumped directly into canals or laterals and used on the lands of the project.

On a part of the area where the pumped waters have not been regarded as needed for supplementing the gravity supply, they have been sold to be used on outside lands. The following tabulation gives the quantities pumped by years, from 1919 to 1926:

Year.	Acre-feet.
1919–20	55 833
1920–21	55 368
1921–22	109 211
1922–23	152 464
1923–24	167 000
1924–25	309 703
1925–26	251 432

All or practically all this pumped water was used for irrigation to supplement the gravity supply.

The system of drainage by wells has an advantage over any system of drains for water recovery. A heavy draft can be made on the ground-water supply during the irrigation season, or during years of short gravity supply, and pumping can be discontinued during the non-irrigation season, or when pumped water is not needed. By this method of operation the water-table can be maintained at all times at a safe distance below the surface and, at the same time, the underground storage supply can be largely conserved.

DISCUSSION

R. I. Meeker,* M. Am. Soc. C. E. (by letter).—The question of return-flow water from irrigation is recognized by engineers as a large factor in river supplies and it is beginning to be appreciated generally by water users on river systems in all the Western States.

Uncertainties concerning dependence on seepage water from irrigation as a large factor in water supplies, long ago passed from the field of speculation in Colorado, where pronounced return-flow waters from irrigation have developed; where seepage or return-flow measurements were initiated in 1885;† and where such measurements have been systematically made to date (1928).‡ The increasing volume and magnitude of such waters are a revelation to the water users in the more recently irrigated areas of the North Platte Valley. If head-gate diversions were a total loss to the North Platte River—as used in water supply studies in 1914—the then threatened interstate water shortage of three States would have become a certainty by this time. Present measurements of return-flow or seepage waters in the North Platte Valley, together with recent engineering data on net water requirements of irrigated lands in older irrigated valleys, are sufficient bases for the assertion that the water supply of the North Platte Basin is sufficient, when controlled by reservoirs, for the needs of three States, with water to spare.

Return-flow measurements and consumptive-use determinations of irrigated lands supplement each other, go hand in hand, and are destined to play a large part in the future irrigation history of rivers. A non-allowance for return-flow water in engineering studies of river supplies will result in a restricted forecast of acreage of lands which may ultimately be served. The beneficial and healing effects of return flow have turned many an irrigation ditch, initiated prior to the set-up of such flow, from failure to success. Return flows from irrigation usually set up slowly and progressively, and may require from ten to twenty years to reach a stabilized condition, dependent on soil conditions, topography, and water usage.

No better illustration of the benefits to be secured from soil storage of water is to be had than that in Fig. 1 on which the natural run-off of the North Platte River is shown, with September and October low flows increased 200 per cent. In recent years the winter flow of this river at Bridgeport, Nebr., has been chiefly return flow.

In the river section between Whalen, Wyo., and Bridgeport, drainage recovery on the North Platte Project has been a large factor in the heavy volume of return flow shown in Fig. 2. At the close of 1927 the drainage system of this project consisted of 350 miles of open-drain ditches costing $1 400 000. This project is a large factor in the heavy seepage and return-flow waters of the North Platte River section from Whalen to Bridgeport. Of the 350 000 acres now irrigated in this section, 170 000 acres are lands of the Federal Project. According to records of the U. S. Bureau of Reclamation, head-gate diversions of the project now average almost 650 000 acre-ft. per year.

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* Cons. Engr., Denver, Colo.

† "Seepage and Return-Flow Waters, Pt. 1; General Discussion and Principles," by L. G. Carpenter, Agricultural Experiment Station, Colorado Agricultural Coll., Fort Collins, Colo.

‡ Biennial Repts., State Engr. of Colorado.

With present information a correlation of head-gate diversions, return-flow data, and consumptive-use determinations of the North Platte Valley between Whalen Dam and Bridgeport, where 350 000 acres are irrigated, shows the following relations concerning water used for irrigation:

Head-gate diversion, 3.50 acre-ft. per acre per year...........	100%
Return flow, 2.30 acre-ft. per acre per year.....................	67%
Consumptive use of river water, 1.20 acre-ft. per acre per year..	33%

With respect to return flow, the irrigated lands of the North Platte Valley in Wyoming and Nebraska are commencing to function in a manner similar to the irrigated lands of the South Platte Valley in Colorado. On account of the more abundant water supply, with consequent lavish use on the irrigated lands, larger return flows per unit of area irrigated are to be expected in the North Platte Valley.

South Platte Valley Return Flow.—The irrigation history of the South Platte Valley in Colorado, furnishes the confirmatory background of what may be expected as to return-flow waters in the North Platte Valley when irrigation conditions become stabilized. Return-flow measurements have been made systematically in the Colorado irrigated areas beginning about 1888. A recent determination of return-flow waters to the channel of the South Platte River (in a distance of 236 miles, extending from a point at the foothills 20 miles above Denver to Julesburg, Colo.) shows a gain of 6 sec-ft. per mile of channel, a total gain of 1 475 sec-ft., which is equivalent to a yearly return of 960 000 acre-ft. These figures apply only to the channel of the river and do not include the return-flow waters of tributary streams, such as Bear and Clear Creeks, St. Vrain, Big Thompson, and Cache la Poudre Rivers, on which large acreages are irrigated. All these streams are recipients of return flow in considerable quantities, and, although such waters are re-diverted and re-used within their own basins, considerable volumes of return flow annually reach the channel of the South Platte River. Such tributary inflow is excluded from river gains.

As an illustrative example of the beneficial and healing effects of return-flow waters, the changed status of the Lower South Platte Valley, from Kersey to Julesburg, is cited: 240 000 acres are now irrigated in this section, as compared with only 100 000 acres in 1902. This region depends almost exclusively on return flow for its water supply. Five reservoirs with a total capacity of 275 000 acre-ft. store and control the winter return flow. In 1902, the Lower South Platte Valley was considered the poorest section of the South Platte River for water supply. To-day, water conditions are just the reverse. In 1925, a low-water year (60% on mountain run-off), the junior ditches and reservoirs of the Lower South Platte Valley received an adequate water supply and the best supply of any section of the South Platte Basin, whereas upper areas, under senior ditches dependent on direct-flow water from the mountains, suffered water shortages.

Table 1 shows in a comparative way the progressive increase of return flow to the main channel of the South Platte River from the head of the irrigated area to the Colorado-Nebraska line, as determined by return-flow measurements covering a period of thirty-five years. The table was prepared from records filed in the office of the State Engineer of Colorado.

TABLE 1.—RETURN-FLOW WATER TO MAIN CHANNEL, SOUTH PLATTE RIVER, PLAINS AREA, EASTERN COLORADO, FROM PLATTE CANYON TO JULESBURG, COLORADO, 236 MILES OF RIVER CHANNEL.

Year.	Total second-feet.	Acre-feet per year.
1891	580	375 000
1900	880	570 000
1908	1 200	775 000
1916	1 410	915 000
1926	1 480	1 000 000

Evidence of the large return flows of the South Platte Basin and consequent re-use of water may also be best indicated by the statement that with an average inflow of mountain water of 1 500 000 acre-ft. per year, 1 100 000 acres are irrigated, and still the unused residue passing to Nebraska is 400 000 acre-ft. per year. A large return flow from soil storage with re-diversion and re-use several times is the answer.

Literature on Return Flow.—The following literature on return flow is cited for convenience of engineers who have need of information on this important subject:

"Seepage or Return-Flow Waters from Irrigation", by L. G. Carpenter. *Bulletin 33,* State Agricultural Coll., Fort Collins, Colo., 1896.

"Seepage Water of Northern Utah", by Samuel Fortier. *Water Supply Paper No. 7,* U. S. Geological Survey, 1897.

"Irrigation in Colorado", by C. W. Beach and Porter J. Preston. *Bulletin 218,* U. S. Dept. of Agriculture, Office of Experiment Stations, Washington, D. C., 1910.

"Irrigation in Arizona", by R. H. Forbes. *Bulletin 235,* p. 51, U. S. Dept. of Agriculture, Office of Experiment Stations, Washington, D. C., 1911.

"Seepage and Return Waters", by L. G. Carpenter. *Bulletin 180,* Pts. 1, 2, and 3, Agricultural Experiment Station, Colorado Agricultural Coll., Fort Collins, Colo., 1916.

Biennial Reports, 10th, 11th, 13th, 14th, and 17th, State Engineer of Colorado; also, unpublished data.

"Irrigation in Northern Colorado", by R. G. Hemphill. *Bulletin 1026,* p. 10, U. S. Department of Agriculture, 1922.

"Return Flow Water from Irrigation Developments", by R. I. Meeker. *Engineering News-Record,* July 20, 1922, pp. 105-108.

"Return of Seepage Water to the Lower South Platte River in Colorado", by R. L. Parshall. *Bulletin 279,* Agricultural Experiment Station, Colorado Agricultural Coll., Fort Collins, Colo., 1922.

Division of Water Rights, Pt. IV, 1922, Department of Public Works, State of California, pp. 49, 98, 99, 104-110; Pt. III, 1924, Return Waters—Sacramento and San Joaquin Rivers, pp. 131-139.

Biennial Report, 15th, State Engineer of Wyoming, Supplement B, "Seepage and Return Water in Wyoming Streams During 1920", by Robert Follansbee, pp. 82-96.

FRED H. TIBBETTS,* M. AM. SOC. C. E. (by letter).—When an irrigated valley is generally underlaid by a water-bearing formation of substantial vertical dimensions and readily available porosity ("specific yield"), and particularly when such a valley is irrigated in large part from reservoir releases such that the accumulation of impounded water is subject to the great annual variation of arid sections, economic studies will then probably show that full development will indicate extensive occasional use of the underground formation for cyclic storage. This general hypothesis is steadily becoming evident in regions like the Salt River Valley of Arizona and the San Joaquin Valley of California. Deep percolation will gradually fill the water-bearing formation with water which, especially if the outflow at the lower end is blocked or retarded, will be protected over a period of years from the large evaporation losses of surface reservoirs, and will then be available during seasons of great water shortage when, by pumping from wells, it can be recovered and used. In such cases it will quite probably be found to be more expensive to distribute the water (including the pumping lift) from the underground reservoir than from surface reservoirs. However, inasmuch as the increase in cost is for power for pumping, this item, felt only during the infrequent cycles of particularly dry years, will not affect greatly the average cost of water.

It is likely that an ultimate balance on large sized projects of from 400 000 to 600 000 acres, will be found when from 10 to 30% of the total water supply is obtained by pumping. The Salt River Valley, the San Joaquin River Water Storage District, and the Pine Flat Storage District (Kings River, California) would be included in this class. In cases of emergency when surface reservoir supplies fail there will be available the possibility (by pumping down the water plane) of furnishing a large part of the required water in this way. Pumping operations of this kind may be perfectly feasible, especially in occasional emergencies, if the aggregate depth of available water-bearing formation within the limits of about 300-ft. wells, is as much as 40 ft. of gravel or sand in which the specific yield under field conditions is about 15 per cent. In such a case a total of 3 ft. of water per year is theoretically procurable, for two successive years from this source alone.

As a further contribution to the subject of return water the writer wishes to present briefly a highly specialized case which apparently violates all the rules of orthodox irrigation engineering in that it seems to demonstrate that the more water used the better for everybody.

The rice industry in California, starting in 1912 with 1 400 acres, increased in 8 years to 162 000 acres. In 1919, the rice crop was valued at more than

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* Civ. Engr., San Francisco, Calif.

$21 000 000. Although rice growing is the greatest of cereal industries, it has nowhere else been introduced quickly on arid lands requiring large quantities of irrigation water with a relative shortage of cheap labor. In California, where labor is scarce and costly, Oriental methods must be modified so as to require a minimum labor cost in preparing and tilling the land, and particularly in irrigating and harvesting the crops. One method of accomplishing this result is to allow the diversion of excessive quantities of irrigation water. Experience has shown that on the areas best adapted to rice culture, the encouragement of such excessive diversions does not lead to the wastage of water and hence the impairment of irrigation water resources.

Analysis of return water from rice irrigation has the simplicity of a textbook problem, because all the water diverted can be completely and accurately accounted for. On three typical projects in the Sacramento Valley—one irrigating about 40 000 acres of rice, another about 10 000, and another about 900 —the irrigation diversions have been measured since 1919. The waste or excess water from these projects is almost immediately collected in drainage canals and returned to the Sacramento River, and again is accurately measured within a few miles of its diversion point. In the typical rice operations of Reclamation District 108, in Colusa Basin, where about 10 000 acres are under irrigation, water is diverted from the river by pumping or by gravity, depending on the river stage. It is then returned to the river within 18 miles of the diversion point in the same manner. Fig. 4 shows for each year since 1919, the average diversion per acre irrigated, segregated into the "consumptively used" water and the water immediately returned to the river. The point that the writer wishes to emphasize is that the consumptive use of the water, and hence the quantity extracted from the river, is practically independent of the quantity diverted. The net extraction is equivalent to a depth of about $5\frac{2}{3}$ ft., and whatever the diversion from the river the remainder, in excess of $5\frac{2}{3}$ ft., is immediately returned.

The drainage water is concentrated in deep drainage canals so that the surface of the water in the canals is generally below the ground-water level during the irrigation season, and always so when adjacent to the rice fields where the land is kept continually submerged. Rice irrigation should be, and in the main is, restricted to heavy soils such as clays and clay-adobes, in which loss of water by deep seepage and percolation is negligible. In soils of these types the seepage is excessively slow. Furthermore, the distance through which the drainage water is transported is so small that evaporation losses are negligible. The result is a clear-cut condition in which the drainage water, or return seepage water, is essentially the diverted water, less the water used in irrigation. The latter represents evaporation losses in the beginning of the season, and when the rice plants are high enough to shade the soil, it represents principally transpiration losses through the growing plants. The average consumption of water from these causes is shown in Fig. 5 for the irrigation season of April, May, June, July, August, and September, in inches per month. This is much in excess of the normal evaporation losses from a free water surface.

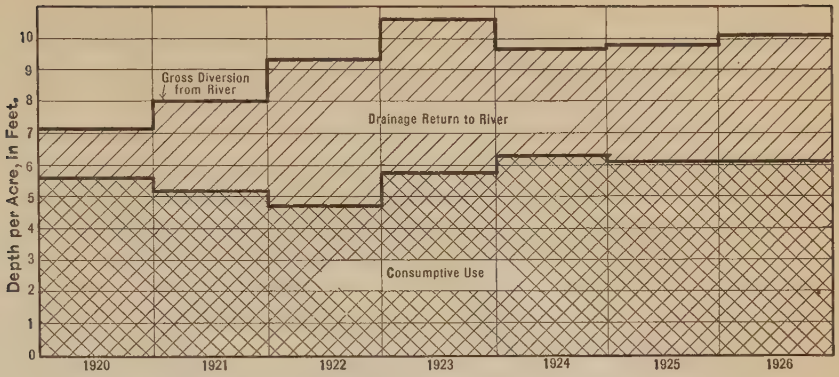

FIG. 4.—ANNUAL DIVERSION, USE, AND RETURN, RECLAMATION DISTRICT NO. 108.

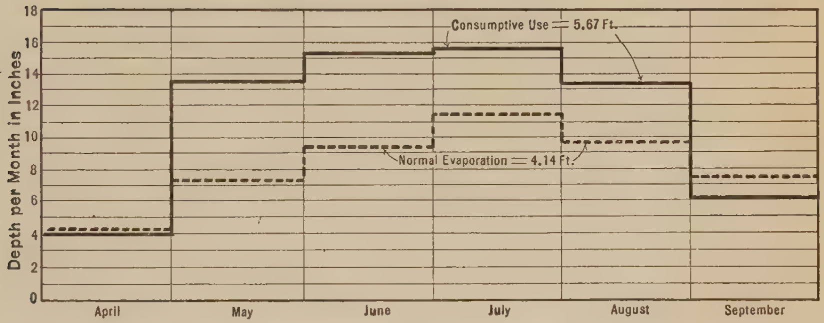

FIG. 5.—AVERAGE CONSUMPTIVE USE AND NORMAL EVAPORATION.

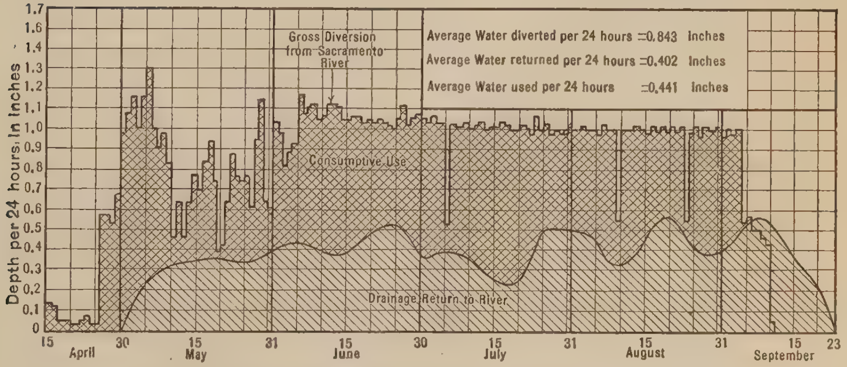

FIG. 6.—TYPICAL SEASONAL DIVERSION, USE, AND RETURN.

One of the advantages of rice irrigation in California is that it makes possible the utilization of lands formerly considered of little value because of alkali. Even if in the first place the lands were not alkaline, it is clear that keeping them submerged to a shallow depth during the hot summer months would tend to draw up alkali from the subsoil. Careful chemical determinations over a period of years in District 108 indicate that in the quite soft Sacramento River water, the 5 acre-ft. used, contains about 2 000 lb. of alkali ("alkali" being popularly taken as the amount of material in solution). If river diversion is restricted so that waste water is less than 15%, alkali accumulates on the land, but if the waste is equal to about 30% of the diversion, that is, if the waste is equal to one-half the water consumed, then from two to four times as much alkali is removed in the waste water, as is applied in the irrigation water. The actual removal of alkali in one instance was at the rate of 9 750 lb. per acre per year.

An ample water supply permits the use of large irrigating heads and large rice checks, with a substantial reduction in labor cost for preparing the soil and distributing the water. Free flow through large checks keeps the water fresh, and tends to maximum yields and early maturities.

In 1920 when an alarming shortage of water was impending in the Sacramento River, State water masters were restricting river diversions as much as possible and the net extraction from the river in District 108 was 5.7 ft., the quantity returned being only 1.6 ft. per acre irrigated. Exactly the same net extraction from this part of the river was made in the season of greatest use in 1923 when the gross head-works diversions exceeded those of 1920 by 3.3 ft. and the quantity returned exceeded the return of 1920 by exactly the same amount. (See Fig. 6.)

These investigations lead to a conclusion radically different from that usually advocated in orthodox engineering, namely, that it is desirable for the industry affected to use excessive quantities of water, and, furthermore, that such excessive use is not a public injury because the available water supply is not diminished thereby. Quantities of water diverted, in excess of the normal consumptive use of about $5\frac{2}{3}$ ft., are immediately and with certainty returned to the stream for re-use.

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Paper No. 1731

TRANS-MOUNTAIN WATER DIVERSIONS*

A SYMPOSIUM

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With Discussion by Messrs. A. Lincoln Fellows, Ralph Bennett, B. F. Jakobsen, Paul M. Entenman, and H. A. Van Norman.

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* Presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927.

INTER-WATER-SHED DIVERSIONS BY TUNNELS ON UNITED STATES RECLAMATION PROJECTS

By E. B. Debler*, M. Am. Soc. C. E.

Synopsis

Trans-mountain diversions, from the Colorado River drainage area to adjacent basins, for irrigation and other uses, have received much consideration in connection with the proposed construction of the Boulder Canyon Reservoir and the proposed compact for the allocation of the Colorado River waters. The aggregate total of such diversions is expected to exceed, in the ultimate, 500 000 acre-ft. per year. One of the most important of these is the Strawberry Tunnel Project, constructed by the U. S. Bureau of Reclamation to irrigate lands in the vicinity of Provo, Utah. Another trans-mountain diversion, although not diverting away from the Colorado River Basin, is the Uncompahgre Tunnel, between Uncompahgre and Gunnison Rivers in Colorado, serving the Uncompahgre Project. This 6-mile tunnel, at the time of its construction (beginning in 1904) was the longest tunnel in the United States, and the most imposing undertaking up to that time in connection with irrigation.

The Gunnison Tunnel

History.—The Uncompahgre River is a tributary of the Gunnison River which, in turn, enters the Colorado River at Grand Junction, Colo. The lower portion of the Uncompahgre traverses a broad fertile valley without sufficient rainfall for crop production and with local stream flows inadequate for the irrigation of the valley. Paralleling this valley and separated from it by a mountainous ridge lies the Gunnison River (Fig. 1) with a flow very much greater than that of the Uncompahgre and devoid of bordering irrigable areas capable of using material portions of its flow. Furthermore, the Gunnison River lies at uniformly higher levels than the Uncompahgre Valley.

Rapid settlement of the valley started in 1884, and numerous large irrigation canals were constructed before stream discharge data had become available. It was soon found that with 100 000 acres under irrigation less than 30 000 acres could reasonably be expected to receive an adequate water supply from local sources.

The diversion of Gunnison River water to the valley was conceived in 1890 by Mr. L. C. Lauzon. In 1894, surveys were undertaken by popular subscription. In 1901, the Colorado State Legislature appropriated $25 000 for the location and construction of a Gunnison Tunnel but, in 1902, these funds were exhausted after a tunnel had been driven 900 ft. In 1901, investigation of the Gunnison Tunnel plan was undertaken by the U. S. Geological Survey under the direction of Mr. A. Lincoln Fellows, the work in 1902 being transferred to the newly created Reclamation Service. A new tunnel location was adopted in 1904, following extensive surveys.

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* Engr., U. S. Bureau of Reclamation, Denver, Colo.

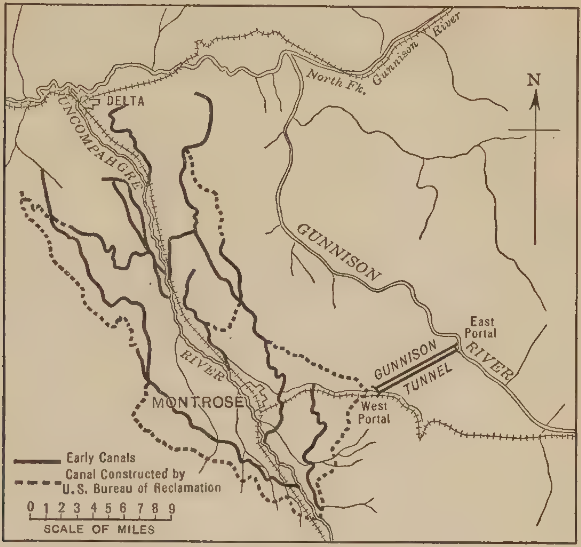
DELTA
UNCOMPAHGRE
North Fk. Gunnison River
N
GUNNISON
East Portal
GUNNISON TUNNEL
RIVER
West Portal
MONTROSE
RIVER
Early Canals
Canal Constructed by U.S. Bureau of Reclamation
0 1 2 3 4 5 6 7 8 9
SCALE OF MILES

Fig. 1.—Map of Gunnison Tunnel and Uncompahgre Project, Colorado.

Plans and specifications for the tunnel were completed in July, 1904, and in the October following, a contract was awarded the Taylor-Moore Construction Company, of Hillsboro, Tex. The contractor withdrew from the work in the following May, after completing about 4.3% of it. In August, 1905, an advertisement was issued for bids on the completion of the work, but those received were considered excessive. The work was then undertaken by the Reclamation Service with plant and equipment turned over by the defaulting contractor, with subsequent additions thereto. The tunnel was holed through on July 6, 1909, and enlarged to full section in February, 1910.

Tunnel Section and Materials.—The tunnel has a length between portals of 5.8 miles. In general, the interior section presents vertical side-walls, 11 ft. apart, surmounted by an arch of 6 to 8 ft. radius, with the crown 12 ft. above the floor. The principal variations during construction related to changes in the radius of the arch to care for changing conditions.

The materials encountered, starting from the west portal, were as follows:

For 2 000 ft., alluvial deposits of sand, gravel, and clay, all water-bearing and very heavy.

For the next 1 200 ft., the same materials with a gradually rising bed of hard shale requiring careful timbering to support the roof and blasting to remove the bench.

The following 10 000 ft., black shale containing inflammable and explosive gases, but dry.

Then 2 000 ft. of badly faulted and shattered mixtures of sedimentary rocks and coal containing gases, and much hot and cold water.

Finally 15 380 ft. of metamorphic granite, gneiss, and schist of greatly varying hardness, with frequent seams carrying much water.

Construction Plan.—The general plan of construction in the alluvial formations was by means of a 4 by 6-ft. top entry, followed by the opening of the arch haunches and sides, with subsequent removal of the bench. In hard formations a top heading, 6 to 8 ft. in height and of full width, was followed by benching.

Timbering was required on about 18 000 ft., or 60% of the tunnel length, almost evenly distributed between ground materials requiring immediate timbering and harder shales permitting a lag of several days behind the bench excavation.

The concrete lining was largely completed by June, 1910, with some lining placed in 1912, and small amounts subsequently. The original plans contemplated full lining, but actually only the floor is wholly lined, with the sides and top lined throughout somewhat more than one-half the length. The tunnel costs, including portals and portal cuts, were roughly $2 900 000, including construction roads and engineering, or about $500 000 per mile.

Utilization.—With full concrete lining as originally contemplated, the estimated capacity of the tunnel was to be 1 300 sec-ft. With the partial lining thus far installed, the maximum diversion has been 975 sec-ft. The original estimated area of irrigable lands in the project was, roughly 140 000 acres. Modifications of the planned distribution system, following the acquisition of numerous existing canals and their adaptation to the project in preference to the construction of new canals, together with seepage developments and the inferior quality of some of the lands originally included, leaves an area of 95 000 acres of good irrigable lands, of which 65 000 acres are now irrigated (1927).

The principal impediment to extension of the irrigated area is widespread seepage without sufficient general interest by the land owners for the adoption of a comprehensive drainage program. For the irrigated area, the delivery of water to the land has averaged 5.7 acre-ft. per acre annually. With a moderate improvement in the duty of water, the combined water supply available from the Uncompahgre River and through the Gunnison Tunnel in its present condition, is estimated to be sufficient for the irrigation of about 85 000 acres.

An increase in the tunnel capacity would not be reflected in a corresponding increase in water supply, as the present capacity is in many years greater than the flow of Gunnison River for varying periods in late summer. The Gunnison has no storage control, but an excellent reservoir site on Taylor Fork has been withdrawn from entry and set aside for use by the Uncompahgre Project in case it should become desirable to increase the low-water flow of the stream in connection with an increase in the tunnel capacity by the completion of the lining.

The annual diversions through the Gunnison Tunnel, all used for irrigation, have averaged 237 000 acre-ft. for the 5-year period ending with

1926. The construction charge payable to the United States for the Gunnison Tunnel and for the construction and reconstruction of canals and laterals is $52 per acre. The principal crops are alfalfa, sugar-beets, and potatoes, with an average annual crop production at the farm of $44 per acre over the past ten years.

The Gunnison Tunnel is not properly a trans-mountain diversion as the term is usually understood, in that the irrigated area lies along a tributary of the parent stream from which the primary water supply is derived. It is, however, to date (1927), the longest tunnel built for irrigation purposes.

STRAWBERRY TUNNEL

History.—The Strawberry Valley Project (Fig. 2) comprises Strawberry Reservoir with a capacity of 255 000 acre-ft. located on Strawberry River, thirty miles easterly from Provo, Utah; the Strawberry Tunnel diverting reservoir waters westerly through the Colorado River-Great Basin Divide into the Spanish Fork water-shed; and a distribution system for lands in the vicinity of Springville, Spanish Fork, and Payson, Utah.

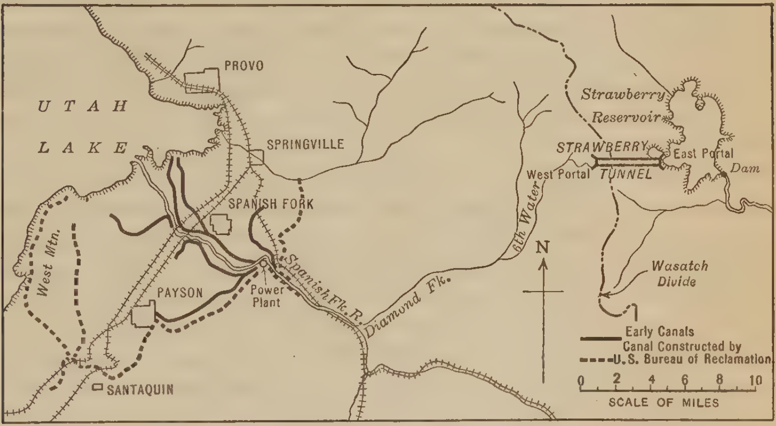
PROVO
UTAH
LAKE
SPRINGVILLE
SPANISH FORK
West Mtn.
PAYSON
Power Plant
Spanish Fk. R.
SANTAQUIN
Diamond Fk.
6th Water
N
Strawberry Reservoir
STRAWBERRY TUNNEL
West Portal
East Portal
Dam
Wasatch Divide
Early Canals
Canal Constructed by U.S. Bureau of Reclamation
0 2 4 6 8 10
SCALE OF MILES

FIG. 2.—STRAWBERRY TUNNEL, STRAWBERRY PROJECT, UTAH.

The diversion of Strawberry River waters to the Spanish Fork Basin received active consideration by the State Engineer of Utah and by irrigators in the Basin about 1900. In 1902, a full survey of the plan was made by a company interested in irrigation storage and power development, but was rejected as too costly.

In 1903, an investigation of the plan was undertaken by the U. S. Reclamation Service at the request of interested citizens, but a number of factors slowed up progress. Among these were the location of the Strawberry watershed within the Uintah Indian Reservation; the extremely low run-off for a few years immediately preceding, indicating a comparatively high construction cost for the estimated yield of water; and lack of adequate interest by land owners to be benefited.

With the definite abandonment of plans to use Strawberry River waters within the Duchesne water-shed, the opening of the Uintah Indian Reservation, and increasing interest in irrigation, a petition for further consideration of the project was received in January, 1905, from about 1 200 citizens. Following the completion of arrangements for repayment of construction expenditures, commencement of work was authorized in March 6, 1906, and $1 250 000 was set aside for this purpose from the Reclamation Fund. Work was immediately started on roads and telephone lines to the tunnel portals.

In August, 1906, bids were invited for the construction of the tunnel, but as no proposals were received, construction by force account was authorized in September, 1906, and promptly commenced at the West Portal. From July, 1907, to November, 1908, work was suspended awaiting the building of a hydro-electric power plant on the lower reaches of Spanish Fork River to furnish power for tunnel construction. Thereafter work was resumed at the West Portal. In October, 1911, work was commenced at the East Portal. The tunnel was holed through on June 20, 1912, and the concrete lining was completed in November, 1912.

Tunnel Data.—The tunnel has a length of 3.6 miles, with an interior section of a flat floor, vertical sides, 7½ ft. apart and 6 ft. high, to the springing line of an arch 2 ft. high. The tunnel is lined throughout. Practically the entire length was through limestone and sandstone of varying hardness, shale being found for a short distance. The cost of the tunnel, including the lining and appropriate charges for hydro-electric power, was $1 150 000, or $320 000 per mile.

Construction Difficulties.—Severe winter conditions hampered construction to some extent, as snowfall often reached 200 in. during the season and temperatures down to 49° Fahr. below zero have been recorded at the East Portal. The principal difficulty in construction work, however, was water which was encountered about midway of the tunnel with a flow of more than 10 sec-ft. and which has continued at 7 to 10 sec-ft. ever since. Swelling shale and limestone wrecked a considerable part of the lining, so that heavy repairs became necessary by 1919, but no trouble has since been experienced.

Utilization.—Strawberry Reservoir, with a capacity of 255 000 acre-ft. available for diversion through the tunnel, is formed by an earth-fill dam with a maximum height of 72 ft. and a length of 488 ft., across Strawberry River, and an auxiliary dam, 37 ft. high and 1 300 ft. long, across a low pass into the Indian Creek water-shed. Early investigations indicated that the cost of collecting water from streams on the north into the reservoir would be excessive in relation to the obtainable water. Indian Creek has been intercepted by a short canal, and an extension thereof to intercept Trail Hollow and other small streams can be made at moderate cost when sufficient demand develops for the water.

The safe annual yield of water from the Strawberry Reservoir is estimated at 80 000 acre-ft., with a possible increase to about 95 000 acre-ft. after complete development. The average draft on the reservoir from 1921 to 1926, inclusive, has been 84 400 acre-ft., which, with 6 000 acre-ft. per year from tunnel leakage, has been added to the flow of Spanish Fork River, averaging 114 000 acre-ft. annually, for diversion by canals serving about 60 000 acres of land at the southeast border of Utah Lake.

The first canals from Spanish Fork River were constructed in 1851 by Mormon settlers. Long before 1900 serious shortages of water were common, with occasionally disastrous results from over-development of the natural flow of the stream. Large storage reservoir sites were not available on the water-shed, and storage of surplus waters on the stream would have infringed on irrigation rights in Utah Lake. The delivery of foreign waters thus solved a problem that might otherwise have resulted in endless litigation.

Water from the Strawberry Reservoir has been sold to the old canals at an average rate of about $45 per acre-ft. of annual use delivered in the stream. It was purchased for an average use of 1 acre-ft. of water for each acre to be irrigated, such water being supplemental to the natural flow. It has been customary in connection with the old canals to use such storage water on about one-half the farm area, the remainder of the area receiving a partial supply only and being devoted to appropriate crops.

The Federal Government constructed two new canals, the High Line Canal for the irrigation of about 20 000 acres of new lands lying above the older lands southerly from Spanish Fork River, and the Mapleton Lateral, primarily to deliver waters to Hobble Creek as a supplemental supply for the canals diverting therefrom. Under the High Line Canal, the average diversion is about 2½ acre-ft. per acre annually. This very high duty for the inter-mountain territory is influenced by the high cost of the water, construction charges on this project being $60 per acre.

Approximately, 10 000 acre-ft. of water from Strawberry Reservoir remain unsold at this time. The present price is $60 per acre-ft. delivered in the stream, with the demand rather light, as the principal areas which could profitably use the water also require drainage and the combined costs deter development.

The principal crops on the older lands of this project are canning vegetables, sugar-beets, and fruits. The newer lands are more largely devoted to alfalfa and grain. The average annual crop value for the project is $45 per acre.

The Strawberry Tunnel diversion is the largest quantity drawn from the Colorado River water-shed; in fact, it constitutes roughly 70% of all such diversions now (1927) being made.

THE PROPOSED LOS ANGELES-COLORADO RIVER AQUEDUCT

By H. A. Van Norman,* M. Am. Soc. C. E.

Synopsis

This paper discusses the need, in Southern California cities, for water from the Colorado River, which is the only large unappropriated supply near this district.

A description is given of several of the proposed routes that would ultimately divert 1 500 cu. ft. per sec. Two of these routes are unique in being gigantic pumping schemes, either of which would utilize a large block of power from the proposed Boulder Dam. Two others—gravity routes—are unusual in that they entail the driving of tunnels of heretofore undreamed of lengths.

Surveys have been extended into 18 000 sq. miles of territory in the Colorado desert previously unmapped.

Various types of aqueduct construction are discussed, as well as the economic problems related thereto.

A Needed Improvement

The recently proposed Los Angeles-Colorado River Aqueduct to provide a municipal water supply from the Colorado River for the City of Los Angeles, and such other cities of Southern California as care to join with her in this enterprise, is unique in two respects. It is to be of greater carrying capacity than any previously constructed aqueduct, and it is not to be in its entirety of the usual gravity type.

The City of Los Angeles now has a population of 1 230 000, and has been growing at a rate which has averaged more than 70 000 inhabitants per year since about 1917. A careful investigation of its present water resources shows that it will reach the limit of its supply by 1937.

Also, in the vicinity of Los Angeles are many suburban cities, some of which are increasing in population even faster, relatively, than that city itself. Some of these cities even now (1927) feel the pinch of water shortage, and many of them see the limit of their supply within a comparatively short time.

In order that these smaller cities may participate in the Los Angeles-Colorado River Project, an Act providing for the formation of Metropolitan Water Districts for the purpose of developing, storing, and distributing water for domestic purposes, was recently passed by the Legislature of the State. Under this Act such a district may be formed of the territory included within the corporate boundaries of any two or more municipalities.

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THE COLORADO RIVER SUPPLY

Unlike most other cities of its size, Los Angeles is at a great distance from any large source of water supply. Realizing this fact, and knowing the time it would take to construct an aqueduct from any distant source, William Mulholland, M. Am. Soc. C. E., Chief Engineer and General Manager of the Water Department, whose duties require him to look into problems of this nature, began, in 1922, to study the sources from which an additional supply might be obtained.

The Colorado River is practically the largest dependable supply situated at a distance that is not prohibitive in point of cost, and available with the fewest legal and engineering difficulties. Its distance from Los Angeles is similar to that of the Owens River, from which the city completed its first aqueduct about 1912.

The unappropriated surplus flow of the Colorado River is more than sufficient to meet the domestic needs of all the cities of Southern California. Unfortunately, this river in its lower reaches is situated at about the same general elevation as most of the territory to be served, and intervening between it and the Pacific Coast, lies a high mountain range, and a great area of relatively high desert. It is impractical to tunnel either in its entirety, and consequently, if the cost is to be kept within reasonable limits, the proposed aqueduct must be constructed at a level higher than its source. The problems to be solved are, therefore, mainly those of engineering.

With these general facts in mind, a party of engineers, headed by Mr. Mulholland, left Los Angeles on October 29, 1923, for a reconnaissance of the territory as far as the Colorado River, in order to investigate the feasibility of the project. Information obtained on this trip showed not only that an aqueduct could be constructed from the river to Los Angeles, but also that several routes were available.

Much of the country traversed consisted of vast desert waste land, separated by rugged mountain ranges acting as barriers. The greater part of the desert section was unmapped, and it soon became apparent that surveys would have to be made on an extensive scale. On the return of the party, machinery was set in motion for perfecting an organization, and making the necessary preliminary surveys.

SURVEYS

The geodetic control was based on the Texas-California arc of primary triangulation, of the U. S. Coast and Geodetic Survey. Such points of the Geological Survey as were available in the region, were used, and a system of triangulation control was adopted to extend the work into the new territory. North American datum for position and mean sea level for elevation were adopted as standards for this survey.

The grid system used by the U. S. Coast and Geodetic Survey for the polyconic projection of maps, was taken as a basis for platting. The One Hundred and Sixteenth Meridian, lying about midway between the Colorado River and the Pacific Coast, was taken as the meridian of reference for map work.

All the topographic mapping has been done by the plane-table method. With experienced topographers, many being former Geological Survey men, more than 18 000 sq. miles of territory have been mapped on scales of 5 000 and 10 000 ft. to the inch, and almost 1 000 sq. miles on the much larger scale of 1 000 ft. to the inch. A contour interval of 100 ft. was adopted for the small scale maps, and intervals of 20, 10, and 5 ft. for those of larger scale.

As a result of these extended surveys, the Department of Water and Power of the City of Los Angeles has constructed a large relief map of all this territory. On it may be delineated practically every route worthy of consideration for the proposed aqueduct.

BLYTHE ROUTE

A route originating 15 or 20 miles above Blythe, Calif., is the one favored by Mr. Mulholland. This route has many attractive features, and most of the construction involved can be done with little difficulty. It has been surveyed and examined in greater detail than the others.

Commencing with an intake approximately due east from Los Angeles, and at practically the nearest point on the river to the city, it skirts the southern end of the Maria Mountains in Riverside County, California, for a distance of from 10 to 15 miles. Then, by successive lifts and grade conduits, alternately, the line would be carried to the divide between the Colorado River Basin and the Coachella Valley of California. This divide is locally known as Shaver's Summit.

The total difference in elevation to be overcome in reaching Shaver's Summit is about 1 400 ft., to which must be added friction head. This would be accomplished in the first 75 miles of aqueduct. In order to overcome this difference in head with the least construction and operating difficulties, the lifting will be done in five stages. From Shaver's Summit to the end of the route, no further pumping will be required.

Between Shaver's Summit and Los Angeles, it will be necessary to construct approximately 35 miles of concrete-lined tunnels along the southwesterly face of the Little San Bernardino Mountains. There will also be a long tunnel under San Gorgonio Pass, varying from 13 to 27 miles in length, depending on its exact location, as yet not fully determined. From its westerly portal, a grade conduit and tunnels will complete the line to Los Angeles.

Along the Colorado River and parallel with the easterly part of this route are extensive gravel deposits, from which, it is believed, a sufficient quantity of clear water for the first few years of operation of the aqueduct can be obtained, either by constructing a large infiltration channel below the water-plane, or by pumping from wells suitably placed, or by both.

The adequacy of these gravels as a preliminary source of supply is to be thoroughly tested by pumping from a 2-mile section of full-sized infiltration canal, and from wells drilled at various points along the river front. About three-fourths of the length of the canal has been excavated to date (1927), and twelve wells have been drilled to depths varying from 60 to 200 ft.

As a result of the construction of a high dam either at Black or Boulder Canyons, it is believed that when a greater supply is needed, the river will be sufficiently desilted at the proposed point of intake to permit of its being pumped directly into the aqueduct. If, however, this is not the case, it can readily be made clear and potable by the usual methods of mechanical filtration.

Black Canyon Route

A route originating at the Black Canyon Dam site has been surveyed in some detail. It is about 85 miles longer than the Blythe route, and involves greater construction difficulties. It also requires that the proposed Boulder Canyon Dam be constructed at the alternative Black Canyon site. As yet, the final location for this dam has not been definitely determined.

All lifts along this route would be near Black Canyon, and the corresponding total head would be greater than that required on the Blythe route. However, considerable return power could be generated at the westerly end of the aqueduct, making the net power consumed in pumping somewhat less than on the preceding route.

One inverted siphon, 13 miles in length, would be required. If this were constructed as a steel pressure line at the ground surface, it would be under a maximum head of 660 ft. If constructed as a tunnel with shafts, the head of course, would be greater. The advisability of a tunnel siphon is questionable, because of the volcanic character of the rock formation. The cost of this route would be materially higher than that of the Blythe route.

Routes for Gravity Supply

In addition to the routes just described, two gravity routes have been suggested by other engineers. One has its point of intake about 110 miles up stream from the Boulder Canyon Dam site, and the other leaves the Colorado River a few miles below its junction with the San Juan, in Utah.

The former requires a dam at Bridge Canyon about 825 ft. in height above its foundation, in order to divert water into its aqueduct. Such a dam would create a reservoir storage of 6 200 000 acre-ft., and this reservoir would have to remain full at all times. The route would be 370 miles in length, including 170 miles of tunnels, with one tunnel of 71 miles, and another of 80 miles. An inverted siphon across the Colorado River near Topock, Calif., having a length of 12 miles and a maximum head of 1 300 ft. would also be necessary. This siphon would have to be constructed as a multi-barrel structure, in order to reduce the thickness of steel plate. If this route were adopted, an additional dam at the Glenn Canyon Dam site near Lee's Ferry, Ariz., would be required before many years had elapsed, to be utilized in postponing the time when the Bridge Canyon Reservoir would become filled with silt.

The other suggested gravity route would be more than 750 miles in length, and contain an even greater mileage of tunnels than the one described. It would remain on the northerly and westerly side of the Colorado River throughout its entire length. It is also proposed that this route be used for service other than domestic water supply in the States of Utah, Nevada, and California.

Sufficient investigation has been made of both of these gravity routes to show not only that they would be extremely expensive to construct, but that the time element would delay the delivery of water to the coast cities of Southern California so long as to make it impracticable as a municipal or metropolitan water project. Both routes contain power recovery features, which would offset, in part, their great cost. At the Bridge Canyon Dam site, in particular, field investigations and measurements create a doubt as to the advisability of constructing a dam to a greater height than the top of the exposed igneous rock, or 630 ft. above the low-water stage of the river. This would reduce the capacity of the reservoir to 3 100 000 acre-ft., and thereby greatly shorten its useful life as a de-silting medium.

Economies and Costs

All possible modifications of location, gradient, or lift, along the non-gravity routes have been under survey. Before the final determination of the particular route to be constructed is made, the economics of the situation require that much thought be given to many complex elements. Among them may be mentioned the following: The length of construction period, in years; the estimated cost of the proposed route completed; the cost as possibly modified by the omission of certain structures, such as units of pumping plants and force mains, which do not have to be constructed to full capacity at the start; the time factor introduced in the construction of the longest or most difficult tunnel, and the number and depths of shafts required; the number of lifts to be installed, and the lengths, sizes, and number of force mains comprising these lifts; the effective head available for return power; the cost of installing pumping plants, power plants, force mains, and pen-stocks; and the annual charges and pumping costs to be met in operating the routes under study.

The estimated cost of any of the several routes will vary with the hydraulic gradient selected. Those with flatter slopes, and correspondingly reduced velocity of water with increased cross-section, obviously will cost more to build. The cost of operating the pumping plants, on the other hand, will decrease with a reduction in the hydraulic gradient; consequently, these latter costs must be balanced in each case against interest charges on the proposed construction cost.

In operating an aqueduct of this character, the main item of cost is that of the power necessary for pumping. To date (1927), this cost has not been fixed, although for the purpose of carrying on economic studies, it has been assumed that power will be available at the rate of 3 mills per kw-hr. This is an estimated price for which power can be sold at the switchboard of a plant situated either at the Boulder or Black Canyon Dam sites, with a dam constructed 600 ft. in height above the low-water level of the river. In all studies due credit is given to an estimated possible income derived from the sale of return power at the westerly end of the proposed aqueduct, wherever head is available for that purpose.

As to the quantity of water required on the completion of the aqueduct project, studies have been made regarding the rate of growth of the population of all such cities of Southern California as may reasonably be expected to participate in the benefits. Their domestic needs have been carefully studied, and after deducting their present supply, the deficiencies have been assumed as being taken from the Colorado River.

The cost of pumping this water, both annually and in total throughout the years, together with the interest charges and installments of principal that will have to be borne by the population to be served, has been computed as carefully as such data can be. The resulting figures are being applied to each proposed route as the field information becomes available.

Aqueducts and Dams

Various types of aqueduct construction are proposed. The quantity of water ultimately to be diverted is 1 500 cu. ft. per sec. When on hydraulic grade the line will be in excavation and completely covered throughout its length. When below hydraulic grade it will be either steel or reinforced concrete pipe, with possibly a pressure tunnel under San Gorgonio Pass. All tunnels will be lined with concrete.

The type of construction proposed for grade conduits is similar to that used on the Catskill Aqueduct. Where the nature of the ground surface makes it advisable to dip below the hydraulic grade and locate under light pressure, a reinforced concrete, hydro-static chord type similar to the 18-ft. conduit of the Ontario Power Company may be adopted. Under heavier pressure, steel pipe lines will be used. Due to the frequency of cloudbursts on the desert area to be traversed, the adoption of either an open conduit or a so-called cut-and-cover section blocking cross-drainage is not considered advisable. Pumping plants, pressure mains, and inverted siphons will be constructed in units as required.

Owing to the steepness of the slopes prevailing in the coastal plain of Southern California, there are few large reservoir sites available for regulating and distributing purposes at that end. The City of Pasadena has a proposed reservoir in San Gabriel Canyon, near Azusa, Calif., which, if constructed, would store 65 000 acre-ft. of water. The Los Angeles County Flood Control District also contemplates the construction of a reservoir near San Dimas, Calif., in what is known as the Puddingstone Reservoir site. It would be capable of storing at least 16 000 acre-ft., in addition to that required for flood-control purposes. Both these proposed reservoirs are situated at elevations which would make them practical for storage and distributing purposes, and all studies made thus far, contemplate maintaining the grade line at such elevation as to make use of either, if available.

Taking all these elements into consideration, the economics of the situation dictate that the route, gradients, and pumping lift to be adopted, be such as to place the least burden on the population to be served. Recognition must be given to the fact that the cities of Southern California are now growing rapidly, and will continue to grow at a rate which may become even faster.

Power Requirements

In conclusion, the construction of any non-gravity aqueduct from the Colorado River to Los Angeles and neighboring cities, is predicated on the utilization of a large block of power. The only source which could furnish power in a sufficient quantity and at a cost not to be prohibitive, is the Colorado River. In order to generate this power, it would be necessary to construct a high dam on the river. The most favorable location for such a dam is either at Boulder Canyon or Black Canyon.

The investigations outlined in this paper have been made under the general supervision of William Mulholland, M. Am. Soc. C. E., Chief Engineer and General Manager, and the writer as Assistant Chief Engineer and General Manager of the Bureau of Water-Works and Supply of the City of Los Angeles. E. A. Bayley, M. Am. Soc. C. E., was in full charge of all field engineering.

TRANS-MOUNTAIN DIVERSIONS IN COLORADO

By Robert Follansbee,* M. Am. Soc. C. E.

Present Development

More than one-half the 103 658 sq. miles comprising the area of Colorado are within the Rocky Mountain region. The eastern part of the State lies within the province of the Great Plains, which extend from the front range of the Rockies to the Missouri River. Irrigation has been practiced in this region for many years, and the area of irrigable land is limited only by the water supply. The sources of this supply are the South Platte and Arkansas Rivers and their numerous tributaries, which rise on the eastern slope of the Rocky Mountains. With the present supply the limit of irrigation on the plains has almost been reached (except for the increased re-use of return seepage water).

In the mountainous region, except for certain relatively small areas, the water supply is in excess of that required by the irrigable lands. This situation has led water users to divert water from that region to the head-waters of the streams on the eastern slope. Table 1 summarizes the description of these diversions.

It will be noted from Table 1 that with the exception of the Laramie-Poudre Tunnel, which is 11 306 ft. long, all the diversions are made by relatively short open ditches at high elevations, and intercept the run-off from small areas.

Future Development

The eleven developments made to date (1927) represent practically the limit of trans-mountain diversions by the relatively inexpensive open ditches, except for the proposed extension of Grand River Ditch. Additional developments must be of greater magnitude and expense, and be made by tunnels and collection ditches leading to them.

The need for additional water on the eastern slope has led water users to propose development from the Fraser, Williams, Blue, and Eagle Rivers and the Fryingpan Creek Basin, the only areas situated so that diversions from them to the eastern slope can be seriously considered for many years to come. Of these, the chief are the Fraser and Blue River diversions. They are on a scale so large, that they can only be undertaken by a municipality like Denver, where the economic phase of the question is not so much a governing factor as the necessity for obtaining additional water to provide for the city's future needs.

It, therefore, appears entirely probable that when these projects are finally undertaken and carried to completion, the systems will be so planned and constructed that they will collect about 80% of the available water. In fact, the city's engineers charged with planning its future water supply are con-

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TABLE 1.—TRANS-MOUNTAIN DIVERSIONS IN COLORADO.

Ditch.	Elevation of diversion, in feet.	Drainage area intercepted, in square miles.	Diversion.		Conduit.			Annual diversion, in acre-feet.
			From.	To.	Length of tunnel, in feet.	Length of ditch, in miles.	Capacity, in second-feet.	
			COLORADO RIVER BASIN.					
Grand River	10 200	12	Colorado River	Cache la Poudre River	0	8	220	11 400
Ewing	10 500	5	Eagle River	Arkansas River	0	5	20	1 910
Berthoud	10 300	4	Fraser River	Clear Creek	462	4	53	685
Cochetopa	10 000	20	Cochetopa Creek	Rio Grande	0	2	40	1 000
Boreas Pass	11 500	2	Blue River	South Platte River	0	1	10	600
			NORTH PLATTE RIVER BASIN.					
Skyline	9 300	14	Laramie River	Cache la Poudre River	0	5	200	16 700
Sand Creek	10 000	...	" "	" " " "	0	10	200	2 900
Laramie-Poudre Tunnel	8 570	67	" "	" " " "	11 306	12.5	800	6 530
Bob Creek	9 800	2	" "	" " " "	0	2	......	840
Rist and McNab	10 300	4	Michigan River	" " " "	0	2.5	50	3 400
Cameron Pass	10 300	1	" "	" " " "	0	2	10	300
Total	......	...	......	......	11 768	54	1 603	46 300
Total for Colorado River	......	...	......	......	462	20	343	15 600

templating an efficiency of 90 per cent. To obtain this high efficiency, the collection ditches in earth and loose rock must be lined, and drainage ditches must be built along the upper side to collect the surface run-off and carry it to the cross-drainage channels which will empty into the main ditch. Also, provision must be made to clean the snow from the ditches by machines, early in the spring.

Judging from the experience gained from the operation of the Grand River and Skyline Ditches, it will be possible to exclude, during the winter months, the run-off from the intercepted streams entering the main ditches, and thus prevent the formation of any great quantity of ice. Also, it may be necessary to cover some parts of the ditches. On the capacities of the tunnels and ditches will depend the storage required. Ditches on southern slopes will obtain water fully two weeks earlier than those on northern slopes; the peak flow will be higher and its subsidence more rapid. Open ditches at lower elevations will encounter less slide rock and will be subject to less danger from heaving, due to the less severe frost action.

The period of the year for which diversion can be made, is taken as that from April 1 to September 30 because that is the average period during which the proposed diversions (except the relatively small one from Fryingpan Creek) can be made without interference with the right of the hydro-electric plant at Shoshone, on the Colorado River. In determining the available water supply no consideration has been given the conflict with water rights immediately below the proposed diversions, as these must be acquired and extinguished.

Fraser River.—With the completion of the Moffat Water Tunnel, which parallels the railroad tunnel at a distance of 75 ft., and has an elevation of 9 300 ft. at its mid-point, the City of Denver has planned to construct a system to divert water to the South Platte Basin. From the west portal of the water tunnel, in NE. ¼ Sec. 10, T. 25 N., R. 75 W., two collection ditches are to be constructed, one 27 miles long, reaching West St. Louis Creek, and the other, 9 miles long, reaching North Ranch Creek. This system will intercept the run-off from 107 sq. miles of drainage area, ranging in altitude from 9 300 to 12 500 ft. From the records of the Fraser River near West Portal (1911-26), the unit run-off for the period from April 1 to September 30 is estimated as 1 060 acre-ft. per sq. mile, or 113 000 acre-ft. from the entire area intercepted. With an efficiency of 80% the mean annual diversion would be 90 400 acre-ft. A study of the 30-year record of the Colorado River near Palisades indicates that the mean for that period is 94% of the 16-year mean (1911-26). On this basis the mean annual diversion would be reduced to 85 000 acre-ft.

The annual variation in the proposed diversion is shown by Table 2, based on the annual percentages for Fraser River, near West Portal, and the mean diversion of 90 400 acre-ft. It will be noted from Table 2 that in the lowest year of the 16-year period, the diversion would have been 63 200 acre-ft.

To carry the maximum discharge of the wettest year, a tunnel capacity of 3 100 sec-ft. would be required. However, it is possible to obtain some

TABLE 2.—ANNUAL DIVERSION FROM FRASER RIVER BASIN.

Year.	Run-off above diversion points, in acre-feet.	Percentage of mean.
1911	83 000	92
1912	108 000	119
1913	66 800	74
1914	119 000	132
1915	108 000	119
1916	82 200	91
1917	84 100	93
1918	117 000	129
1919	63 200	70
1920	80 400	89
1921	106 000	117
1922	66 800	74
1923	86 800	96
1924	81 300	90
1925	75 900	84
1926	105 000	116
Mean	90 400	....

storage above the proposed diversions, and this would reduce the required tunnel capacity. A capacity of 1 000 sec-ft. would have required annual storage during the past 16 years, as follows:

Year.	Acre-feet.	Year.	Acre-feet.
1914	22 200	1921	15 400
1915	25 000	1923	3 880
1917	3 990	1924	3 990
1918	26 900	1926	3 590

Williams River.—Several surveys have been made to investigate the diversion of water from Williams River to Clear Creek in the South Platte Basin. The most comprehensive survey has been made by the City of Denver, which proposes to bore a tunnel 3 miles long at an altitude of 10 300 ft.* The west portal will be on Bobtail Creek, a tributary of Williams River, and the east portal on the West Fork of Clear Creek. From the west portal, one collection ditch will extend to an unnamed creek beyond McQueary Creek, a distance of 4 miles, and another ditch will extend to the South Fork of the Williams River, a distance of 18 miles. These ditches will intercept the run-off from 29 sq. miles of drainage area, ranging in altitude from 10 400 to 12 500 ft. No records of the Upper Williams River are available, but by a comparison with the Fraser River records, the unit run-off above 10 400 ft. from April 1 to September 30, inclusive, is estimated as 1 100 acre-ft., or 32 000 acre-ft. for the entire 29 sq. miles. With an efficiency of 80%, the mean annual diversion would be 25 600 acre-ft. A tunnel capacity of 600 sec-ft. will be required without storage on the western slope to divert this quantity of water.

A study of the Palisades records indicates that the 30-year mean is 94% of the 16-year mean (1911-26). On this basis, the mean annual diversion would be reduced to 24 000 acre-ft.

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* This is about the lowest altitude believed to be feasible for a water tunnel, as the slopes on both sides of the Continental Divide are such that a tunnel at 9 300 ft., the altitude of the Fraser River diversion, would be 11 miles long.

Blue River.—This basin is so situated with reference to the eastern slope that a number of possible diversions present themselves. Surveys have been made by the City of Denver for diversions at elevations of 10 300, 9 500, 9 112, and 8 842 ft.; and the writer has made a preliminary study for one at an elevation of 9 800 ft. Table 3 shows the possibilities at each elevation proposed.

TABLE 3.—PROPOSED DIVERSIONS FROM BLUE RIVER AT VARIOUS ELEVATIONS.

DIVERSION.		Elevation, in feet.	LENGTH OF CONDUIT.		Area intercepted, in square miles.	Mean annual diversion (80 %), in acre-feet.
From.	To.		Tunnel, in miles.	Ditch, in miles.		
Snake River.....	West Geneva Creek	10 300	3	26	56	28 000
Illinois Gulch...	Michigan Creek.........	9 800	7.5	25	100	57 000
Swan River......	Halls Gulch.............	9 500	12.9	26	181	96 000
Blue River	North Fork, South Platte River..........	9 112	18.9	14	293	189 000
" "	North Fork, South Platte River..........	8 842	22.8	1	328	197 000

The quantity of water that can be diverted will depend on the length of the collection ditches built from the receiving portal of the tunnel along the mountain sides. Since the diversion of a given quantity of water requires a shorter collection ditch as the elevation of the tunnel is decreased, it follows that, for the same length of collection ditch, the quantity of water that can be diverted, will increase with the decrease in elevation.

In Table 3 it will be noted that for 26 miles of ditches the diversions would increase from 28 000 to 96 000 acre-ft., with an increase in tunnel length from 3 to 12.9 miles. The projects requiring tunnel lengths of 18.9 and 22.8 miles make available such large diversions that a study showing those available with ditches 26 miles long has not been made. Since the City of Denver is the only water user that can afford to make a diversion from Blue River, at least within the period of time for which a forecast of future use can safely be made, and since a tunnel 12.9 miles long will yield a quantity of water sufficient for Denver's needs for the same period, and is within the realm of present-day engineering practice, the diversion of 96 000 acre-ft. at an elevation of 9 500 ft. has been selected for the purpose of this paper.

The west portal would probably be on Swan River, in SE. ¼ Sec. 15, T. 6 S., R. 77 W., 1 mile below Tiger. Collection ditches extending from the North Fork of the Snake River on the north to North Barton Creek on the south would have a total length of 26 miles and intercept the run-off from 181 sq. miles of drainage area ranging in elevation from 9 500 to 13 500 ft. Stream-gauging records are available for the Blue River, at Dillon (1911–26) and for the Snake River, at Dillon (1911-19). For the nine-year period (1911-19), the mean run-off from April 1 to September 30 is 148 000 acre-ft., or 676 acre-ft. per sq. mile for 219 sq. miles, ranging in elevation from 8 850 to 13 500 ft. As the run-off increases with elevation, the unit run-off for the area above 9 500 ft. will be slightly greater, and has been taken as 718 acre-ft. This gives a total run-off of 120 000 acre-ft. above the 181 sq. miles lying above the collection ditches. With an efficiency of 80%, the mean annual diversion will be 96 000 acre-ft.

A study of the 30-year record of the Colorado River near Palisades indicates that the mean for that period is 95% of that for the 9-year period covered by the Blue and Snake River records. On this basis the mean annual diversion from Blue River would be 91 200 acre-ft.

The variation in run-off available for diversion from April 1 to September 30 each year under the project, at 9 500 ft. elevation, is shown by Table 4 which is based on a mean run-off of 96 000 acre-ft., and an annual percentage determined from the run-off from April to September each year at the gauging station on Blue River, at Dillon.

TABLE 4.—ANNUAL DIVERSION FROM BLUE RIVER BASIN.

Year.	Run-off above diversion points, in acre-feet.	Percentage of mean.
1911	82 500	86
1912	115 000	120
1913	86 500	90
1914	134 000	140
1915	82 500	86
1916	85 500	89
1917	106 000	111
1918	111 000	116
1919	69 000	72
1920	87 300	91
1921	121 000	126
1922	72 000	75
1923	111 000	116
1924	87 200	91
1925	72 000	75
1926	119 000	124
Mean	96 000	....

It will be noted that in the lowest year of the 16-year period, the diversion would have been 69 000 acre-ft.

The maximum discharge at the tunnel site during the years of record was 1 880 sec-ft. With a tunnel capacity of 1 000 sec-ft., the following storage would have been required to care for the excess run-off:

Year.	Acre-feet.
1914	19 500
1917	6 480
1918	17 000
1921	10 700
1923	2 050
1924	1 930
1926	6 740

Eagle River.—At Tennessee Pass, which separates the Eagle and Arkansas Drainage Basins, an opportunity is afforded for the diversion of water from Eagle River. A tunnel at an elevation of 10 200 ft. would be 2 miles long and at 9 800 ft., 6 miles long. Since the slope of Eagle River Basin prevents the interception of any area comparable in size with the areas in the Fraser and Blue Drainage Basins, it is evident that for the quantity of water available, a length of tunnel greater than 2 miles would not be warranted. With the tunnel at 10 200 ft. elevation and 15 miles of collection ditches reaching Bennett Creek on the west side, and East Fork on the east, an area of 30 sq. miles will be intercepted, ranging in elevation from 10 200 to 13 000 ft. Records of Eagle River at Redcliffe from 1911 to 1925 show the mean run-off from April 1 to September 30 to be 594 acre-ft. per sq. mile. For the area above 10 200 ft. elevation, the unit run-off is estimated as 700 acre-ft. per sq. mile, or 21 000 acre-ft. for the 30 sq. miles intercepted. With an operating efficiency of 80%, 16 800 acre-ft. could be diverted.

A low divide between Eagle River and Tenmile Creek Drainage Basins makes possible a diversion at 10 900 ft. by a tunnel ¼ mile long.* A ditch, 3½ miles long, on the west side reaching Searles Gulch, and one, 6½ miles long, on the east side, reaching Mayflower Gulch, will intercept an area of 20 sq. miles, ranging in elevation from 10 900 to 13 500 ft. Records of Tenmile Creek, at Dillon, from 1911 to 1919, show the mean run-off from April 1 to September 30 to be 872 acre-ft. per sq. mile. For the area above 10 900 ft., the unit run-off is estimated as 1 000 acre-ft. per sq. mile, or 20 000 acre-ft. for the entire area intercepted. With an efficiency of 80%, 16 000 acre-ft. could be diverted. The diversion from Tenmile Creek would be made to the East Side Collection Ditch of the Eagle River System.

The annual variation in the combined diversions proposed from the Eagle and Tenmile Basins is shown in Table 5, which is based on a mean discharge of 32 800 acre-ft. adjusted according to the variation at the Eagle and Tenmile Gauging Stations.

TABLE 5.—ANNUAL DIVERSION FROM EAGLE RIVER AND TENMILE CREEK.

Year.	Run-off above diversion points, in acre-feet.	Percentage of mean.
1911	39 000	119
1912	42 600	130
1913	25 600	78
1914	41 300	126
1915	23 300	71
1916	31 800	97
1917	33 800	103
1918	41 300	126
1919	24 300	74
1920	33 400	102
1921	41 300	126
1922	23 000	70
1923	34 400	105
1924	27 600	84
1925	22 700	69
Mean	32 800	......

It will be noted in Table 5 that in the lowest year of the 15-year period, the diversion would have been 22 700 acre-ft. Without storage on the western slope, a tunnel capacity of 700 sec-ft. would be required.

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* Lowering the elevation of the tunnel 100 ft. would increase its length to 1½ miles.

From the Palisades records it appears that the 29-year mean (1897-1925) is 93% of the 15-year mean (1911-1925), and, on this basis, the mean run-off available for diversion would be 30 500 acre-ft.

Fryingpan Creek.—Ivanhoe Tunnel, in Sec. 13, T. 9 S., R. 82 W., 11 miles west of Leadville, on the abandoned Colorado Midland Railway, affords an opportunity to divert water from Fryingpan Creek to Lake Fork, a tributary of the Arkansas River. The tunnel, which is at an altitude of 10 950 ft., is 2 miles long, with an eastward slope of 15 ft. per 1 000. Near the west portal, lies Lake Ivanhoe which can be converted into a reservoir of 1 920 acre-ft. capacity by a 30-ft. dam at each end. A collection ditch, 14 miles long, heading in the main fork of Fryingpan Creek, would intercept the run-off from 12 sq. miles of drainage area, ranging in altitude from 11 000 to 13 500 ft. Records of Fryingpan Creek at Norrie (Elevation 8 440), from 1911 to 1916, show the mean run-off from April 1 to September 30 to be 1 040 acre-ft. per sq. mile. For the area above 11 000 ft., the unit run-off is estimated to be 1 300 acre-ft. per sq. mile, or 16 000 acre-ft. for the 12 sq. miles intercepted. At an efficiency of 80% the annual diversion would be 12 800 acre-ft. Irrigation interests in the Arkansas Valley have started construction work on this diversion.

Summary of Future Development.—The developments here described are summarized in Table 6.

TABLE 6.—SUMMARY OF FUTURE DEVELOPMENT.

DIVERSION.		Drainage area intercepted, in square miles.	Elevation of tunnel, in feet.	LENGTH OF CONDUIT, IN MILES.		Annual diversion, in acre-feet.
From.	To.			Tunnel.	Ditch.	
Fraser River........	South Boulder Creek.	107	9 300	6	36	90 400
Williams River.....	West Clear Creek.....	29	10 300	3	22	25 600
Blue River..........	South Platte River....	181	9 500	12.9	26	96 000
Eagle River.........	Arkansas River........	50*	10 200	2	15	32 800
Fryingpan Creek...	" "	12	10 950	2†	14	12 800
Total............	......	379	......	25.9	113	258 000

* Including diversion from Tenmile Creek.

† Ivanhoe Tunnel on abandoned Colorado Midland Railway.

From the magnitude of the work involved, it will probably be many years before these diversions are completely developed.

Effect of Diversions on the Flow of the Colorado River.—To determine the effect of the possible diversions on the flow of the Colorado River at various points, it is necessary to compute the monthly quantities to be diverted. For that purpose, the period, 1911-26, in so far as available for each stream, was selected, and the monthly percentages for the entire discharge from April 1 to September 30 were determined from the records on each stream. These percentages were applied to the mean diversions for each project.

TABLE 7.—MEAN MONTHLY DISCHARGE OF PROPOSED DIVERSIONS.

Drainage basin.	Discharge, in Acre-Feet.					
	April.	May.	June.	July.	August.	September.
Fraser River...........	2 710	17 200	40 700	18 100	7 220	4 520
Williams River.........	1 020	4 850	11 000	5 370	2 050	1 280
Blue River.............	3 840	19 200	36 500	21 100	9 600	5 760
Eagle River............	1 970	9 520	12 500	5 240	2 290	1 300
Fryingpan Creek	640	3 070	5 380	2 300	900	510
Total..................	10 200	53 800	106 000	52 100	22 100	13 400

The effect of these diversions on the flow of the Colorado River is shown by deducting from the mean monthly recorded flow for the period, 1911 to 1926, in so far as it is available, the computed quantities given in Table 7, and expressing the differences as percentages of the recorded flow. These are given in Table 8 for each principal gauging station above the Green River, and for Yuma, Ariz.

TABLE 8.—DISCHARGE MODIFIED BY PROPOSED DIVERSIONS EXPRESSED AS PERCENTAGES OF MEASURED DISCHARGE.

Gauging station.	April.	May.	June.	July.	August.	September.
Kremmling, Colo.*	91	85	82	80	78	77
Glenwood Springs, Colo.†	93	90	88	85	84	85
Palisades, Colo.‡..........	96	94	93	91	91	92
Cisco, Utah................	98	97	95	94	94	94
Yuma, Ariz.................	99.3	98.0	97.8	98.0	98.0	97.6

* Above diversions from Eagle River and Fryingpan Creek.
† Above diversions from Fryingpan Creek.
‡ Above diversions for Grand Valley.

The effect of the proposed diversions on the flow of the Colorado River will be a maximum at the Kremmling Station and will decrease at points farther down stream where the discharge is successively greater. From the substantial uniformity of annual variation in discharge throughout the Upper Basin, it is believed that the percentages in Table 8 will apply each year.

DISCUSSION

A. LINCOLN FELLOWS,* ESQ. (by letter).—Mr. Debler has furnished a valuable contribution to the literature on trans-mountain diversions which the writer takes pleasure in endorsing as generally correct. However, in the interest of historical accuracy he desires to supplement the statement made with reference to the commencement of work on the Gunnison Tunnel. The author correctly states that the plan was conceived in 1890 by Mr. L. C. Lauzon, but the impression is given to the casual reader that the State of Colorado led the way in commencing work on the construction of the tunnel. This is true in so far as actual construction on a State tunnel was concerned; but, in reality, the U. S. Geological Survey was first, by some months. That organization set apart funds for the commencement of surveys in the latter part of 1900, on the recommendation of F. H. Newell, M. Am. Soc. C. E., and the writer. The appropriation by the State Legislature followed in 1901, five or six months later. Construction of a tunnel by the State was begun after the first preliminary survey was completed, before the final surveys and reports were made, and contrary to the advice of the writer. This State tunnel was entirely distinct and located at a considerable distance from the Government location. It was eventually abandoned completely.

Some very interesting trans-mountain diversions have been made in Wyoming, several of them dating back many years. Indeed, Wyoming has been a pioneer in making such diversions. The writer also calls attention to a notable project in North Dakota—the suggested diversion of the Missouri River into the Red River drainage basin, or, in other words, the proposed diversion from a stream discharging into the Gulf of Mexico to the Hudson Bay drainage basin. The State Irrigation Engineer of North Dakota, Robert E. Kennedy, has prepared preliminary reports regarding this project.

RALPH BENNETT,† M. AM. SOC. C. E. (by letter).—Only those routes for the proposed Los Angeles-Colorado River Canal that lie north of the Imperial Valley, are mentioned specifically by Mr. Van Norman.

There are also a number of possible routes in which the water would be diverted through the Imperial Canals and carried in them for varying distances before entering the plants that would lift the water over the range (see Fig. 3).

Of these, the most southerly, and, therefore, the ones of particular interest to San Diego, Calif., would divert from the existing Imperial System in the vicinity of Dixieland and would rise to the crest at an elevation of nearly 4 000 ft. above sea level in less than 40 miles. Even with power at 0.5 cent per kw-hr., water so delivered would cost San Diego less than that secured by the present storage system. As this diversion would deliver into coastal streams which flow into large existing reservoirs of that city, it could be put in without reference to Colorado River storage and the plant could be operated only when excess flow was available.

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* Irrig. Engr., Bureau of Public Roads, U. S. Dept. of Agriculture, Denver, Colo.

† Cons. Engr., Los Angeles, Calif.

The next northerly possibility is a diversion from the Imperial System in the vicinity of Westmoreland into a conduit running west through the Borego Valley and crossing under the summit in a long tunnel under the Town of Julian. This route would vastly supplement the water supply in the vicinity of Oceanside, but would not reach the urban district around Los Angeles to advantage. It would also be deficient in storage.

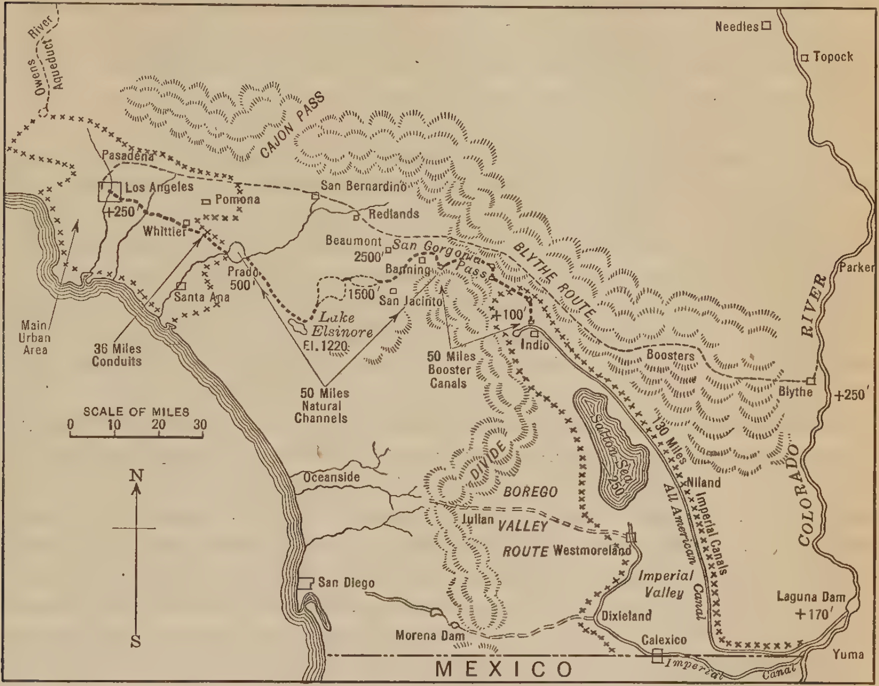
Owens River Aqueduct
Needles
Topock
Pasadena
CAJON PASS
Los Angeles
+250'
Pomona
San Bernardino
Redlands
Whittier
Beaumont 2500'
San Gorgonio Pass
Banning
BLYTHE ROUTE
Prado 500'
1500'
San Jacinto
+100'
Parker
RIVER
Santa Ana
Lake Elsinore El. 1220
Indio
Main Urban Area
36 Miles Conduits
50 Miles Booster Canals
Boosters
Blythe
+250'
SCALE OF MILES
0 10 20 30
50 Miles Natural Channels
DIVIDE
Salton Sea
130 Miles
Niland
COLORADO
N
S
Oceanside
BORREGO
Julian VALLEY
ROUTE Westmoreland
All American Canal
Imperial Canals
Imperial Valley
San Diego
Laguna Dam +170'
Dixieland
Morena Dam
Calexico
Yuma
MEXICO
Imperial Canal

Fig. 3.—Map Showing Los Angeles-Colorado River Aqueduct.

The most northerly of the Imperial Valley routes would lie through the Imperial and Coachella Valleys to a regulating basin west of Indio at about Elevation +100. This point would be almost 130 miles from Laguna Dam across a smooth unbroken plain of soft soils well suited to the excavation of an earth canal. As the revised crest at Laguna will be at least as high as Elevation +170, there would be ample fall available.

In reaching this point, the All-American Canal and its extension northwesterly would serve more than 500 000 acres of land, and incidental to that service the water would be completely desilted before the first pumping plant was reached. From this terminal reservoir of the Imperial System the water would be lifted by boosters through a series of canals lying in the more elevated portions of the valley until it passed over the Beaumont crest of San Gorgonio Pass into the coastal areas. These canals would not require tunnels or extensive rock excavation.

During the first half of the development period, or until at least a flow of 1 000 sec-ft. was in actual use, even the summit lift would be without reduction by tunnel. For these, sections could be located so that power recovery plants could be placed cheaply on the westerly slopes and return fully one-half the gross power. This run in high-level canals and pumping lines would not exceed 50 miles in total length.

The Beaumont summit is at Elevation 2 500 and is 80 miles in an air line from Los Angeles. In the route suggested by Mr. Van Norman this 80 miles is covered by a continuous high-line conduit passing directly through many of the smaller cities en route. However, that route possesses no present or future possibilities for the storage essential to the certain delivery of 1 000 000 acre-ft. of water per year 270 miles from its diversion from a natural channel.

Better economy can be had by spilling at the summit southwest into the San Jacinto River. Along the channel of this stream below Elevation 1 500 and continuing to Lake Elsinore, there is a series of possible storages with a great aggregate possible capacity, and of extraordinary safety since the basin is closed at Elsinore by a natural dike and not by an artificial dam.

For the Los Angeles Metropolitan Supply, at least, the water may again be spilled into the available natural channels and may be recovered only 36 miles from Los Angeles in the narrows of the Santa Ana River at Elevation 500.

This Imperial-San Jacinto route requires a total of only 216 miles of construction, no indispensable tunnels, and less than 50 miles will require lining. The gross power consumption for full flow would be increased 25% over the Van Norman route, but with the same length of summit tunnel it would be only 5% greater. The time of construction would be fixed by the delivery of large pumps and the assembly of canal excavation equipment rather than by the time to drive tunnels. The initial cost would be predicated on the initial flow to be handled since the earth canals can be enlarged as the demand grows.

If, at some later date, a full gravity system is substituted, the loss of fixed investment would be limited to the abandoned pumping plants and such high-level canals as do not fit the proposed new elevation. The earth canals in the Imperial Valley would be released to purely irrigation use.

Tentative estimates indicate that the cost along this line would not be more than one-half or one-third that along the Blythe route; and here, again, as in the San Diego line, the possible coastal or delivery end storage would be so great that the line could be operated regardless of storage conditions on the Colorado River. Indeed it could be operated during some years of light load with off-peak power from the existing systems, or the 70 000 kw. which may be made available from the All-American Canal without reference to the completion of the power plant at the Boulder Canyon Dam.

B. F. Jakobsen,* M. Am. Soc. C. E. (by letter).—Mr. Van Norman states that a careful investigation shows that Los Angeles will reach the limit of its water supply within ten years, but he does not give any authority for this statement. Careful engineering investigations made by competent engineers, such as Thomas H. Means, M. Am. Soc. C. E., Paul Bailey, M. Am. Soc. C. E., and others, show that additional water supply can be obtained from Owens Valley and Mono Basin in sufficient quantity to justify the building of another aqueduct from Owens Valley with a capacity of about 450 sec-ft., and that the power which can be developed would pay for this additional aqueduct. This water is ideal in quality for domestic use, and would supply a population of about 4 000 000. It will most likely be much more than "ten years" before this population is reached.

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* Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

Mr. Van Norman refers to a large infiltration canal. There will be about 8 000 acre-ft. of mud to be removed each year from the 1 500 sec-ft. of flow, for which Los Angeles has applied from the Colorado River. Assume the width of the available river area to be 528 ft., or 0.1 mile, or an area of 64 acres per mile length of infiltration canal. If the silt is deposited in the filter to a depth of 2 ft. and there are 25% voids, 250 miles of infiltration canal per year will be needed. This does not look very practical, and the writer doubts whether the usual methods of mechanical filtration, which is suggested as an alternative, will work much better.

The storage capacity of Bridge Canyon Gravity Aqueduct diversion is given by Mr. Van Norman as 6 200 000 acre-ft., and he states that this would be filled with silt before many years, and that, therefore, if this site were selected, it would require an additional dam within a short time. At this point the river carries about 100 000 acre-ft. of silt annually, so that if all the silt were deposited, it would still take 62 years to fill the reservoir. Long before the silt problem would become serious, other dams would have been built in the natural course of developing the river.

Mr. Van Norman states that the two gravity routes have been investigated and found "extremely expensive" to construct, and that the time element needed for construction make them impracticable. In a recent article he stated* that the annual pumping cost of the proposed project will be $8 400 000. Capitalizing this at 4.5% per year gives $187 000 000. If Mr. Van Norman will add this to the cost of the proposed pumping plan, the writer believes he may find it difficult to maintain his assertion that the gravity routes will be "extremely expensive."

Doubt has been expressed by Mr. Van Norman whether a dam higher than 630 ft. above the low-water stage of the river can be safely built at the Bridge Canyon site. A geological report described the rock above the exposed igneous rock as hard quartzitic sandstone. It might be well to leave this an open question until decision can be made on the most competent geological and engineering advice.

Much value would be added to this paper if Mr. Van Norman would give the detailed cost data which he states have been prepared. It would be especially interesting to compare the two proposed gravity routes with the proposed pumping scheme with its enormous annual pumping charges.

Mr. Van Norman states that the most favorable location for a power dam is at Boulder Canyon. A number of such sites have been described in a

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[*] *Engineering News-Record*, May 31, 1928, p. 853.

report* by E. C. La Rue, M. Am. Soc. C. E., according to which it would be a serious mistake to construct the proposed Boulder Canyon Dam, since it does not fit into a general plan of development of the river, and would result in a waste of about 300 000 h.p. William Kelly, M. Am. Soc. C. E., while Chief Engineer for the Federal Power Commission, also investigated the proposed Boulder Canyon Dam and recommended against it.

Mr. Van Norman has failed to mention the opinions and ideas of competent engineers who differ with him. Some plan should be arranged which would receive the approval of a considerable number of competent engineers, and it should be understood that there is nothing final about the proposed plan.

PAUL M. ENTENMAN,† ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Van Norman's paper shows clearly that, of the routes surveyed for the Los Angeles-Colorado River Aqueduct, there are but two that deserve serious consideration —the Blythe route and the Black Canyon route. As the Black Canyon is predicated on the choice of a site for the construction of the proposed Boulder Dam (which may not be the one finally selected) the adoption of the Blythe route, or a route farther south, seems inevitable. This is apparent since it will be desirable to begin early construction of the aqueduct, whereas the approval of legislation authorizing the Boulder Canyon project may be delayed.

Mr. Van Norman states that investigations of the feasibility of the Blythe route as a project independent of the construction of the Boulder Dam, are being conducted. Without a high dam creating a reservoir that will act as a desilting basin, the problem of excluding the silt of the Colorado River from the aqueduct will be one of major importance.

The proposed intake is located 120 miles below the canyon, in the section of the river characterized by broad, fertile, deltaic plains. In this region, the river flows in a bed of its own alluvium. Logs of wells and other subsurface explorations show that the alluvial deposit is of more or less uniform character for great depths. It is marked by extreme fineness of the constituent particles. Underground flow through this material is so slow that it is doubful whether an infiltration basin of sufficient capacity to serve the aqueduct would be feasible, for irrespective of the porosity of the material in which the infiltration canal may be constructed, flow into the canal must be through the fine bed silts of the river.

The rate of underground flow, investigated with Hazen's well-known formula,‡ assuming $d = 0.10$ mm., is found to be a velocity too low to serve an infiltration canal of reasonable dimensions.

The assumed value of $d = 0.10$ mm. is larger than the effective size of the sand grains in the bed of the Colorado. Fortier and Blaney§ give a mechanical analysis of the deposits in the bed of the river from a number of samples; one, taken at the intake of the Palo Verde Canal, within five miles of the site of the proposed intake for the Los Angeles Aqueduct, shows that 14.62% of the sample consisted of grains having diameters of 0.05 mm.*

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* Report No. 556, U. S. Geological Survey.

† Cons. Engr., Banning, Calif. Mr. Entenman died March 29, 1929.

‡ "Filtration of Public Water Supplies," by Allen Hazen. M. Am. Soc. C. E.

§ "Silt in the Colorado River," by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., *Technical Bulletin No. 67*, U. S. Bureau of Public Roads.

A theory that extensive strata of water-bearing gravels may be found below the silt and sand has considerable currency. The investigations of Fortier and Blaney show clearly that very little material that can be classified as coarse sand reaches the lower river.† The canyon section of the stream is an efficient grinding mill reducing boulders, gravel, and sand to a silt of which more than 50% is finer than Portland cement.

Occasional beds of gravel are found where débris cones at the mouths of washes debouch directly into the river; but even then gravel is not found in extended sheets. During floods, the river has sufficient velocity to transport any gravel or small boulders which may enter it from a wash; and successive floods cause a wide distribution of this débris. Such gravel beds as are found in the Colorado River occur in lenses or in long trains of coarse material in the buried channel of the stream.

As it is unlikely that an infiltration canal of sufficient capacity to serve the proposed Los Angeles Aqueduct will be feasible, the intake structure must include some desilting device. The most successful desilting structure on the Lower Colorado is the settling channel and sluice-way at Laguna Dam. A diversion weir raises the level of the river about 10 ft. The settling basin is 650 ft. long, with a cross-section 126 ft. wide at the top, 116 ft. at the bottom, and 18 ft. deep. At its down-stream end are three sluice-way openings, 33 ft. wide, controlled by Stoney roller gates. Water is skimmed from the surface of the settling channel through a gate with thirty-five 7½-ft. openings regulated by flash-boards.

Once a week, the sluice-gates are opened, and the silt which has accumulated in the settling channel is sluiced back into the river below the Laguna Dam. The settling channel is effective in removing all the bed silt and about 50% of the suspended silt in the river.‡

The site of the proposed intake for the Los Angeles Aqueduct is not favorable for the construction of a weir, but as all the water has to be pumped in any event, head for sluicing purposes could be developed with pumps. However, unless the fluctuations in discharge are to be controlled completely by the construction of the Boulder Canyon Reservoir, some form of diversion structure will be necessary. The experience of the Imperial Irrigation District with rock-fill dams across the Colorado shows that a much cheaper structure than the Laguna Dam, such as that closing the Bee River channel,§ will serve this purpose, provided there is ample spillway capacity to prevent overtopping.

The settling channel for the proposed aqueduct should be designed to reduce the velocity in this basin to less than ½ ft. per sec.; and the gates controlling the flow to the aqueduct should be of such capacity that only the surface water between 6 in. and 1 ft. in depth will be skimmed from the settling channel. Some device other than flash-boards should provide regulation of flow. It is surely not beyond the ingenuity of engineers to design a skimming gate that will be tighter than a flash-board structure. At Laguna Dam, it has been found that some of the silt which enters the canal leaks through and around the flash-boards, and that more silt than is actually suspended in the surface water of the settling basin flows into the canal through the boards and by reason of vertical currents that form along the face of the regulating gates. Laboratory studies of the control of these vertical currents and of the design of a form of hinged gate that would replace the crude flash-boards is warranted for a project of the magnitude of the Los Angeles Aqueduct.

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* "Silt in the Colorado River," by Samuel Fortier, M. Am. Soc. C. E., and Harry F. Blaney, Assoc. M. Am. Soc. C. E., *Technical Bulletin No. 67*, U. S. Bureau of Public Roads, p. 73, Table 51.

† *Loc. cit.*, p. 9.

‡ *Loc. cit.*, p. 89.

§ "A River Diversion on the Delta of the Colorado," by S. L. Rothery, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVI (1923), p. 1412.

The quantity of silt carried by the Lower Colorado River varies with the discharge. At the lower end of the canyons 0.86%, by volume, is an average. When the aqueduct is operated to full capacity, this would mean transporting nearly 10 000 acre-ft. of silt each year. Unless this silt is removed at the intake, its deposition in storage reservoirs along the route of the aqueduct would rapidly diminish their capacity.

The silt should be removed at the river for another reason. Its abrasive nature is destructive of the working parts of pumps. More than 70% is silica, and its exceeding fineness permits its penetrating close spaces. An ordinary centrifugal pump will have its bearings ground out in a month's or six weeks' operation. Pumps are made with outboard bearings to meet this problem, but the runners and shell wear rapidly, especially when operated at high speeds and against high heads.

If the head for sluicing purposes is developed with pumps instead of with a diversion dam, such pumps would have to handle silty water. However, the lift would be comparatively low, and large-capacity, low-lift pumps of the propeller type have handled Colorado River water without excessive wear. That settling channels are completely successful for the removal of silt has been demonstrated on a small scale in the water-works plant at Calexico, Calif.

Any study of the silt problem of the Colorado as it affects a domestic supply for Los Angeles, as well as the irrigation projects in the lower river, emphasizes the importance of constructing some desilting structure in the canyons of the river. The most recent computations of the quantity of silt transported to the end of the canyon section are those of Fortier and Blaney who estimate that 137 000 acre-ft. reach the lower river annually.* They conclude that: "The most feasible and economical means of solving the silt problem * * * is to impound the river silt behind a high dam, such as is proposed at Boulder Canyon."

H. A. Van Norman,† M. Am. Soc. C. E. (by letter).—The writer wishes to thank the discussers for the interest which they have shown in the proposed Los Angeles-Colorado River Aqueduct. Many of the ideas suggested are feasible under some one or more variations of the conditions of the problem which are not specifically defined.

However, several additional steps have been made toward the realization of this gigantic and needed improvement, two of which are worthy of mention.

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* *Technical Bulletin No. 67*, U. S. Bureau of Public Roads, p. 4.

† Asst. Chf. Engr., Dept. of Water, City of Los Angeles, Los Angeles, Calif.

The first step occurred on November 6, 1928, when eleven incorporated cities, in the Southern California Coastal Plain, signified their intention, by popular vote, of joining a Metropolitan Water District of Southern California for the purpose of procuring water from the Colorado River. The second great stride was the passing of the Swing-Johnson Bill on December 21, 1928. The significance of this Bill is that the Federal Government definitely decided its policy in the allocation of water from the Colorado River and made the first step toward development by the authorization of the construction of the Boulder Dam. The location of the first dam to be built on the river necessarily fixes a definite outline for future development. This eliminates one of the heretofore variable conditions in the plan of any Los Angeles-Colorado River Aqueduct.

The construction of a high dam at Black Canyon by the Federal Government, which will completely regulate the flow of the Colorado River and prevent the major portion of the silt from reaching any smaller reservoirs which may be built below it, allows the alternatives not mentioned in the paper, to be considered.

Blythe-Parker Route.—Under this condition, a dam approximately 80 ft. in height might be built at one of two favorable locations above Parker, Ariz. The first site is situated just below the junction of the Williams River and is about 12 miles above Parker. The other site is about 6 miles up stream from Parker. An 80-ft. dam at either place would create a reservoir of approximately 1 000 000 acre-ft., which would raise the water surface to Elevation 440 and re-desilt the water for the aqueduct. From a reservoir so created, a line could be built along the west side of the river to connect with the present Blythe route south of the infiltration gallery. An alternative would come down the river front to the north end of the Riverside Mountains where a pump lift would raise the water to Elevation 940. Thence, it would proceed westerly and southerly, *via* Grommet and Rice, Calif., tunneling through the Coxcomb Mountains, to a junction with the Blythe route, at an elevation of 1 300 ft., about 12 miles east of Shaver's Summit.

Pichaco Route.—This proposed route originates in a reservoir about 30 miles up stream from Yuma, Ariz. The reservoir would be created by a dam of undetermined height to be built in a canyon of the Colorado River, about 3 miles below the old mining town of Picacho, Ariz., where the water surface has an elevation of 170 ft. It would have a capacity of from 1 000 000 to 2 000 000 acre-ft., depending on the height of the dam. Drillings are now (1929) being made to determine to what height it is feasible to construct a dam at this point. Enough power can be generated at the dam to lift the water to Elevation 500.

From the upper end of a pump lift at Elevation 500, the aqueduct pierces the Picacho Mountains in a tunnel approximately 14 miles long, with shallow shafts, to the east slope of the Salton Sea Basin. Thence, it continues northwesterly in conduit through smooth desert country along the eastern slope of the basin to a point east of Mecca, Calif., where there is a 10-mile stretch of short tunnels. Thence, the line proceeds in conduit to a point 6 miles northeast of Indio, Calif., where the water is lifted approximately 1 025 ft. to Elevation 1 250, continuing in conduit section to 1 mile west of Whitewater, Calif., where there is another pump lift of 275 ft. to an elevation of about 1 500, at which point it joins the Blythe route, or a variation thereof. From this point the aqueduct follows the Blythe route under San Gorgonio Pass and to the Metropolitan Water District.

The Picacho route, if constructed, would be about 242 miles long from the Colorado River to a terminal reservoir at or near San Gabriel, Calif., and would require a total lift of approximately 1 580 ft. This route throughout its length would be through accessible country, in the proximity of the Southern Pacific Railroad except at the point of diversion.

The statement by Mr. Jakobsen as to the ultimate life of the reservoir at Bridge Canyon is correct if one assumes 100 000 acre-ft. of silt per annum, and that the full volume is effective as a desilting medium; but if the figure of 137 000 acre-ft. of silt per annum, as given by Fortier and Blaney in their more recent studies, is used, and only a part of the volume of the reservoir is considered for silt storage, then the effective life of a reservoir at Bridge Canyon would be greatly reduced.

Mr. Jakobsen also states that by capitalizing the annual pumping cost which would have to be met on any of the pumping routes, at 4½% per year, the money value would be $187 000 000. This would be nearly cut in half if he would use an interest rate of about 7 to 8%, in order to offset depreciation, operation, and maintenance charges on the excess length and cost of the gravity aqueduct with which he is making his comparison. It would be further reduced if consideration were given to the fact that during the first years of operation the annual pumping costs are less than one-half those under full development of the project.

Mr. Bennett suggests a more southerly diversion from the Colorado River and the joint use of the Imperial Canals. Such a route would have the lowest initial cost of any of those described for the proposed aqueduct; but with the great amount of pumping and the necessity of desilting the water by settling basins or other means, it would be the most expensive to operate and maintain.

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Paper No. 1732

THE STIFFNESS OF SUSPENSION BRIDGES*

By S. Timoshenko,† Esq.

With Discussion by Messrs. Lloyd G. Frost, Hans H. Rode, E. Steuermann, George C. Priester, and S. Timoshenko.

Synopsis

In this paper a method is developed by which the additional cable stress and the deflection curve produced in a suspension bridge either by live load or by temperature variation may be directly calculated. Due to the quick convergence of the series used, the deflections and the bending moments in the span truss can be calculated in much shorter time than by the usual method.

I.—Introduction

The theory of the stiffened suspension bridge originated with Rankine and was developed on the assumption that the suspended truss remains absolutely rigid under the action of live load; further development was made by Professor Melan,‡ who first took into consideration the deflection of the span truss. The application of this theory to the design of large American suspension bridges, such as the Manhattan Bridge, in New York, N. Y., and the Philadelphia-Camden Bridge, showed§ that this more exact theory is of great practical importance; furthermore, it resulted in a considerable economy of material.

In applying Melan's theory to determine the additional horizontal component of the cable stress due to any cause, such as live load or temperature change, the assumption has been made that not only dead load but also additional load may be considered as uniformly distributed along the span. In

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[*] Published in May, 1928, *Proceedings*.

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[‡] This theory is discussed in detail in "The Theory and Practice of Modern Framed Structures," by J. B. Johnson, C. W. Bryan, and F. E. Turneaure, Vol. II, p. 276.

[§] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 919.

this manner an equation has been established from which the additional cable stress can be calculated by a "cut-and-try" method. The assumption of uniform load distribution is sufficiently accurate provided the span is fully loaded, or nearly so; but a considerable error may occur when only a small part of the structure is loaded.*

In this paper a method is developed by which (1) the actual distribution of live load can be taken into consideration; and (2) the additional cable stress produced either by such a load or by temperature variation may be directly calculated. In using this method the deflection curve of the span truss is presented in the form of a trigonometrical series, the terms of which can easily be calculated from a consideration of the energy of the system.

II.—Differential Equations for the Deflection Curve of the Span Truss

Consider first the case shown in Fig. 1.† It is assumed that the initial curve of the cable is parabolic and that the entire dead load is carried by the cable and causes no stress in the stiffening trusses. The truss is only stressed by live load and by changes in temperature from the normal.

Let,

H_w = horizontal component of cable stress, due to dead load and mean temperature.

H = additional horizontal component of cable stress, due to any cause, such as live load or temperature change.

$\beta = \frac{H}{H_w}$.

η = deflection of truss and cable at any point, from the normal position, due to H, plus any given live load (the effect of the stretch of the hangers is neglected).

w = dead load per unit length = load on cable, including its own weight.

p = live load per unit length over a part or all of the span truss.

EI = flexural rigidity of the span truss, assumed to be constant along the span.

q = additional load on the cable, due to live load or temperature change.

f = the sag of the cable.

l = the span of the stiffened truss.

The differential equation of the initial curve of the cable is:

$$H_w \frac{d^2 y}{d x^2} = -w \quad \text{.........................} (a)$$

By integration the known parabolic deflection curve is:

$$y = \frac{4 f x (l - x)}{l^2} \quad \text{.........................} (1)$$

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* "The Theory and Practice of Modern Framed Structures," by J. B. Johnson, C. W. Bryan, and F. E. Turneaure, Vol. II, p. 312.

† The notations and initial assumptions are the same as those contained in "The Theory and Practice of Modern Framed Structures."

When an additional vertical load, q, is acting on the cable, the differential equation of the curve of the cable becomes:

$$(H_w + H)\frac{d^2}{d\,x^2}(y+\eta) = -(w+q) \dots\dots\dots\dots\dots (b)$$

From Equations (a) and (b) the load, q, transmitted to the cable will be obtained:

$$q = -H\frac{d^2}{d\,x^2}(y+\eta) - H_w\frac{d^2\,\eta}{d\,x^2}$$

or,

$$q = \beta\,w - H_w(1+\beta)\frac{d^2\,\eta}{d\,x^2} \dots\dots\dots\dots\dots\dots\dots (2)$$

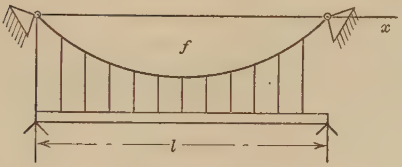

FIG. 1.

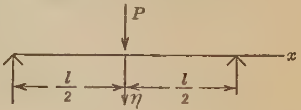

FIG. 2.

The load transmitted to the span truss will be:

$$p - q = p - \beta\,w + H_w(1+\beta)\frac{d^2\,\eta}{d\,x^2} \dots\dots\dots\dots\dots (3)$$

Substituting this in the known differential equation of the deflection curve of a beam of uniform cross-section, the following equation for the deflection curve of the span truss will be obtained:

$$E\,I\frac{d^4\,\eta}{d\,x^4} = p - \beta\,w + H_w(1+\beta)\frac{d^2\,\eta}{d\,x^2} \dots\dots\dots\dots\dots (4)$$

This is a linear differential equation with constant coefficients, which can easily be solved in any particular case provided the load, p, and the change, H, of the horizontal component of the cable stress are known.

III.—PARTICULAR CASE

Assume that a concentrated load, P, is applied at the middle of the span truss (Fig. 2), and that the change, H, in the horizontal component of the cable stress can be neglected in comparison with H_w. Then, $p = 0$, $\beta = 0$, and Equation (4) becomes:

$$E\,I\frac{d^4\,\eta}{d\,x^4} - H_w\frac{d^2\,\eta}{d\,x^2} = 0 \dots\dots\dots\dots\dots\dots\dots (5)$$

The general solution of this equation is,

$$\eta = A\,e^{kx} + B\,e^{-kx} + C\,x + D \dots\dots\dots\dots\dots\dots (6)$$

in which, A, B, C, and D are arbitrary constants, e is the Napierian base, and,

$$k = \sqrt{\frac{H_w}{E\,I}} \dots\dots\dots\dots\dots\dots\dots\dots\dots (7)$$

The arbitrary constants must now be determined from the conditions at the ends and at the middle of the span. At the end, $x = \frac{l}{2}$,

$$(\eta)_{x=\frac{l}{2}} = 0; \quad \left(\frac{d^2\eta}{dx^2}\right)_{x=\frac{l}{2}} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (c)$$

At the middle, the tangent must be horizontal and the shearing force equal to $-\frac{P}{2}$; therefore,

$$\left(\frac{d\eta}{dx}\right)_{x=0} = 0; \quad EI\frac{d^3\eta}{dx^3} = \frac{P}{2} \ldots\ldots\ldots\ldots\ldots\ldots (d)$$

It is easy to show by differentiation that Equation (5) and the conditions, (*c*) and (*d*), will be satisfied by taking,

$$\eta = \frac{P}{2\,k\,H_w}\left(-\frac{\sinh k\left(\frac{l}{2} - x\right)}{\cosh\frac{k\,l}{2}} + k\left(\frac{l}{2} - x\right)\right) \ldots\ldots\ldots (8)$$

from which,

$$\eta_{\text{max.}} = \frac{P}{2\,H_w}\left(\frac{l}{2} - \frac{1}{k}\tanh\frac{k\,l}{2}\right)$$

$$M_{\text{max.}} = -EI\left(\frac{d^2\eta}{dx^2}\right)_{x=0} = \frac{P}{2\,k}\tanh\frac{k\,l}{2}$$

Take, for instance,*

$P = 100\,000$ lb.;
$EI = 29 \times 10^6 \times 43\,900 \times 144$ lb. per sq. in.;
$H_w = 10.48 \times 10^6$ lb.; and
$l = 1\,447.12$ in.

Then,

$$\frac{1}{k} = \sqrt{\frac{EI}{H_w}} = 349 \text{ ft.}$$

$$\eta_{\text{max.}} = 22.0 \text{ in.}; \quad M_{\text{max.}} = \frac{P}{2\,k}\tanh\frac{k\,l}{2} = 0.968\,\frac{P}{2\,k}$$

Comparing these results with the deflection,

$$\frac{P\,l^3}{48\,EI} = 59.5 \text{ in.}$$

and with the maximum bending moment,

$$M_{\text{max.}} = \frac{P\,l}{4} = \frac{P \times 1\,447}{4} \text{ ft-lb.}$$

for a simply supported beam, it can be concluded that due to the support provided by the cable the deflection of the span truss is reduced to about one-third and the maximum moment to about one-half that of an unsupported truss.

For a load, *P*, unsymmetrically applied or a distributed load, *p*, covering only a part of the span, the determination of the arbitrary constants of the general solution, Equation (6), becomes more complicated and a method based on the use of a trigonometrical series for the deflection curve (Section IV) is simpler for numerical calculations.

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* These numerical data represent the middle span of the Manhattan Bridge.

IV.—Application of Trigonometrical Series

The deflection curve of a beam with simply supported ends (Fig. 3) always can be represented in the form of the trigonometrical series,

$$\eta = a_1 \sin \frac{\pi x}{l} + a_2 \sin \frac{2 \pi x}{l} + a_3 \sin \frac{3 \pi x}{l} + \ldots\ldots\ldots\ldots (9)$$

in which a_1, a_2, a_3, . . ., are coefficients which can be calculated from a consideration of the energy of the system.

By using for the potential energy of bending according to the well-known equation,

$$U = \frac{E I}{2} \int_0^l \left(\frac{d^2 \eta}{d x^2}\right)^2 d x$$

substituting from Equation (9) for η; and taking into consideration that,

$$\int_0^l \sin \frac{m \pi x}{l} \sin \frac{n \pi x}{l} d x = 0$$

and,

$$\int_0^l \sin^2 \frac{m \pi x}{l} d x = \frac{l}{2}$$

the potential energy becomes,

$$U = \frac{E I \pi^4}{4 l^3} (a_1^2 + 2^4 a_2^2 + 3^4 a_3^2 + \ldots.)\ldots\ldots\ldots\ldots (10)$$

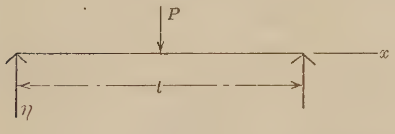

FIG. 3.

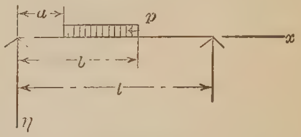

FIG. 4.

Assuming that a small increase, δa_n, is given to one of the coefficients, a_n, of the series in Equation (9), a small additional deflection, $\delta a_n \sin \frac{n \pi x}{l}$, will be superimposed on the deflection given by Equation (9); then, from the condition of equilibrium it can be concluded that the work done by loads acting on the beam during the displacement, $\delta a_n \sin \frac{n \pi x}{l}$, must be equal to the increase of the potential energy, Equation (10). In the case already considered the beam was under the action of a load, P, applied at the middle, and of a distributed load, $H_w \frac{d^2 \eta}{d x^2}$ (see Equation (5)), representing the action of the cable on the beam. The work done during the small displacement, $\delta a_n \frac{\sin n \pi x}{l}$, by the load, P, will be,

$$P \delta a_n \sin \frac{n \pi}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (e)$$

The work done by the distributed load, $H_w \frac{d^2 \eta}{d x^2}$, will be,

$$\int_0^l H_w \frac{d^2 \eta}{d x^2} \delta a_n \sin \frac{n \pi x}{l} d x$$

or, substituting for η from Equation (9), and considering that only the term containing the coefficient, a_n, gives an integral different from zero, the expression for this work becomes,

$$- H_w \delta a_n a_n \frac{n^2 \pi^2}{2 l} \ldots\ldots\ldots\ldots (f)$$

The increase of the potential energy, U, Equation (10), due to the small displacement, $a_n \sin \frac{n \pi x}{l}$, will be :

$$\frac{\delta U_n}{\delta a_n} \delta a_n = \frac{E I \pi^4 n^4}{2 l^3} a_n \delta a_n \ldots\ldots\ldots\ldots (g)$$

Equating Equation (g) to the sum of Equations (f) and (e),

$$\frac{E I \pi^4 n^4}{2 l^3} a_n = - H_w a_n \frac{n^2 \pi^2}{2 l} + P \sin \frac{n \pi}{2}$$

from which, when n is odd,

$$a_n = \frac{2 P l^3}{E I \pi^4} \frac{\sin \frac{n \pi}{2}}{n^4 + n^2 \alpha}$$

and, when n is even,

$$a_n = 0 \qquad \Bigg\} \ldots\ldots\ldots\ldots (h)$$

in which,

$$\alpha = \frac{H_w l^2 *}{E I \pi^2} \ldots\ldots\ldots\ldots (11)$$

Substituting Equation (h) in Equation (9),

$$\eta = \frac{2 P l^3}{E I \pi^4} \left(\frac{\sin \frac{\pi x}{l}}{1 + \alpha} - \frac{\sin \frac{3 \pi x}{l}}{3^4 + 3^2 \alpha} + \frac{\sin \frac{5 \pi x}{l}}{5^4 + 5^2 \alpha} \cdots \right)$$

The deflection at the middle $\left(x = \frac{l}{2} \right)$ will be,

$$\eta_{\text{max.}} = \frac{2 P l^3}{E I \pi^4} \left(\frac{1}{1 + \alpha} + \frac{1}{81 + 9 \alpha} + \frac{1}{625 + 25 \alpha} + \ldots . \right)$$

The series is quickly convergent and very convenient for numerical calculations.

Applying this to the previous numerical example,

$$\alpha = \frac{H_w l^2}{E I \pi^2} = 1.75$$

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*The term, α, has a very simple meaning. It represents the ratio of the horizontal component, H_w, of the cable stress to the column load of the span truss.

and,

$$\eta_{\max.} = \frac{2\,P\,l^3}{E\,I\,\pi^4}\left(\frac{1}{2.75} + \frac{1}{96.9} + \frac{1}{668.7} + \ldots\ldots\right) = 0.376\,\frac{2\,P\,l^3}{E\,I\,\pi^4} = 22.0 \text{ in.}$$

Within the accuracy of the slide-rule this result coincides with the former solution.

This method can also be applied to the general case given by the differential Equation (4), in which the load acting on the truss is given by Equation (3). Taking, again, the series of Equation (9) for the deflection curve and assuming that the live load, p, is uniformly distributed between the limits, $x = a$ and $x = b$, as shown in Fig. 4, the work done by the live load on a small displacement, $\delta\,a_n \sin \frac{n\,\pi\,x}{l}$, will be,

$$p\,\delta\,a_n \int_a^b \sin\frac{n\,\pi\,x}{l}\,d\,x = \delta\,a_n\,\frac{p\,l}{n\,\pi}\left(\cos\frac{n\,\pi\,a}{l} - \cos\frac{n\,\pi\,b}{l}\right)$$

The work done by the reactions from the cable on the truss will be,

$$-\,\delta\,a_n\,\beta\,w \int_0^l \sin\frac{n\,\pi\,x}{l}\,d\,x + \delta\,a_n\,H_w\,(1+\beta)\int_0 \frac{d^2\,\eta}{d\,x^2}\sin\frac{n\,\pi\,x}{l}\,d\,x$$
$$= -\,\delta\,a_n\,\frac{l}{n\,\pi}\,\beta\,w\,(1 - \cos n\,\pi) - a_n\,\delta\,a_n\,H_w\,(1+\beta)\,\frac{n^2\,\pi^2}{2\,l}$$

The equation for determining a_n becomes,

$$\frac{p\,l}{n\,\pi}\left(\cos\frac{n\,\pi\,a}{l} - \cos\frac{n\,\pi\,b}{l}\right) - \frac{l}{n\,\pi}\,\beta\,w\,(1 - \cos n\,\pi)$$
$$-\,a_n\,H_w\,(1+\beta)\,\frac{n^2\,\pi^2}{2\,l} = \frac{n^4\,E\,I\,\pi^4}{2\,l^3}\,a_n$$

from which,

$$a_n = \frac{\dfrac{p\,l}{n\,\pi}\left(\cos\dfrac{n\,\pi\,a}{l} - \cos\dfrac{n\,\pi\,b}{l}\right) - \dfrac{l}{n\,\pi}\,\beta\,w\,(1 - \cos n\,\pi)}{\dfrac{n^4\,E\,I\,\pi^4}{2\,l^3} + H_w\,(1+\beta)\dfrac{n^2\,\pi^2}{2\,l}}$$

or, when n is odd,

$$a_n = \frac{2\,l^4}{E\,I\,\pi^5\,(n^5 + n^3\,\alpha\,(1+\beta))}\left(p\left(\cos\frac{n\,\pi\,a}{l} - \cos\frac{n\,\pi\,b}{l}\right) - 2\,\beta\,w\right)$$

and, when n is even,

$$a_n = \frac{2\,p\,l^4}{E\,I\,\pi^5\,(n^5 + n^3\,\alpha\,(1+\beta))}\left(\cos\frac{n\,\pi\,a}{l} - \cos\frac{n\,\pi\,b}{l}\right) \qquad (12)$$

Substituting from Equations (12) in Equation (9), a quickly convergent series will be obtained from which the deflections can be calculated for any distribution of live load, provided that the additional horizontal component, H, of the cable stress is known.

Assuming, for instance, that the live load covers only one-quarter of the span to the left support of the truss, and substituting $a = 0$ and $b = \frac{l}{4}$ in

Equations (12), the deflection curve is obtained in the form of the following series:

$$\eta = \frac{2\, l^4}{E\, I\, \pi^5}\left(\frac{p\left(1 - \cos\dfrac{\pi}{4}\right) - 2\,\beta\, w}{1 + \alpha\,(1 + \beta)}\sin\frac{\pi\, x}{l} + \frac{p\,\sin\dfrac{2\,\pi\, x}{l}}{2^5 + 2^3\,\alpha\,(1 + \beta)} + \frac{p\left(1 - \cos\dfrac{3\,\pi}{4}\right) - 2\,\beta\, w}{3^5 + 3^3\,\alpha\,(1 + \beta)}\sin\frac{3\,\pi\, x}{l} + \ldots\right) \qquad (13)$$

from which, the deflection can be calculated if $\beta = \dfrac{H}{H_w}$ is known.

For calculating the bending moments the second derivative of Equation (13) must be calculated and substituted in the known equation, $M = -\,E\,I\,\dfrac{d^2\,\eta}{d\,x^2}$.

If there is no live load and if the effects of temperature alone on the deflection are being considered, then from Equations (12), when n is odd,

$$a_n = -\,\frac{4\,\beta\, w\, l^4}{E\, I\, \pi^5\,(n^5 + n^3\,\alpha\,(1 + \beta))}$$

and, when n is even,

$$a_n = 0$$

The deflection curve will then be,

$$\eta = -\,\frac{4\,\beta\, w\, l^4}{E\, I\, \pi^5}\left(\frac{\sin\dfrac{\pi\, x}{l}}{1 + \alpha\,(1 + \beta)} + \frac{\sin\dfrac{3\,\pi\, x}{l}}{3^5 + 3^3\,\alpha\,(1 + \beta)} + \frac{\sin\dfrac{5\,\pi\, x}{l}}{5^5 + 5^3\,\alpha\,(1 + \beta)} + \ldots\right) \qquad (14)$$

The minus sign indicates that an increase in cable stress, $(\beta > 0)$, produces a deflection of the truss in an upward direction.

V.—DETERMINATION OF THE ADDITIONAL HORIZONTAL COMPONENT OF CABLE STRESS

To determine H, the additional cable stress due to live load or to change in temperature, the change in potential energy of the cable must be calculated. The initial tensile force in the cable for any section is,

$$H_w\,\frac{d\, s}{d\, x}$$

and the potential energy of tension in an element, $d\, s$, will be,

$$\frac{\left(H_w\,\dfrac{d\, s}{d\, x}\right)^2}{2\, A\, E}\, d\, s$$

in which, A is the area of the cross-section of the cable.

The increase of this energy, due to an additional horizontal component, H, will be,

$$\frac{(H_w + H)^2 \left(\frac{d\,s}{d\,x}\right)^2 d\,s}{2\,A\,E} - \frac{H_w^{\,2} \left(\frac{d\,s}{d\,x}\right)^2 d\,s}{2\,A\,E} = \frac{H\left(H_w + \frac{1}{2}H\right)}{A\,E}\,\frac{d\,s^3}{d\,x^2}$$

so that the complete change in the energy of the cable will be:

$$\frac{H\left(H_w + \frac{1}{2}H\right)}{A\,E}\int_0^l \frac{d\,s^3}{d\,x^2} = \frac{H\left(H_w + \frac{1}{2}H\right)}{A\,E}\,L \quad\text{..........(15)}$$

in which,

$$L = \int_0^l \frac{d\,s^3}{d\,x^2}$$

Taking as the basis of calculation the parabolic curve given by Equation (1),

$$L = l\left[\frac{1}{4}\left(\frac{5}{2} + \frac{16\,f^2}{l^2}\right)\left(1 + \frac{16\,f^2}{l^2}\right)^{\frac{1}{2}} + \frac{3}{32}\log_e\left(\frac{4\,f}{l} + \left(1 + \frac{16\,f^2}{l^2}\right)^{\frac{1}{2}}\right)\right] \quad\text{................(16)}$$

The increase in energy, Equation (15), must be equal to the work done by the load acting on the cable. The intensity of the initial load is w. The intensity of the final load is (Equation (2)),

$$w + q = w + \beta\,w - H_w\,(1 + \beta)\,\frac{d^2\,\eta}{d\,x^2}$$

Assuming that, for small variations in cable stress, the deflections are proportional to the load, an average value of the intensity of the load must be taken when calculating the work done. Hence,

$$T = \int_0^l \left(w + \frac{q}{2}\right)\eta\,d\,x = \int_0^l \left(w + \frac{w\,\beta}{2} - \frac{H_w}{2}(1 + \beta)\,\frac{d^2\,\eta}{d\,x^2}\right)\eta\,d\,x$$

Substituting from Equation (9) for η,

$$T = \left(w + \frac{w\,\beta}{2}\right)\frac{2\,l}{\pi}\left(a_1 + \frac{a_3}{3} + \frac{a_5}{5} + \ldots\right) + \pi^2\,\frac{H_w}{4\,l}(1 + \beta)\,(a_1^{\,2} + 2^2\,a_2^{\,2} + 3^2\,a_3^{\,2} + \ldots) \quad\text{...........(17)}$$

Equating this to the increase in potential energy of the cable,

$$\frac{H_w^{\,2}\,\beta\left(1 + \frac{1}{2}\beta\right)}{A\,E}\,L = \left(w + \frac{w\,\beta}{2}\right)\frac{2\,l}{\pi}\left(a_1 + \frac{a_3}{3} + \frac{a_5}{5} + \ldots\right) + \frac{\pi^2\,H_w}{4\,l}(1 + \beta)\,(a_1^{\,2} + 2^2\,a_2^{\,2} + 3^2\,a_3^{\,2} + \ldots)$$

or,

$$\frac{H_w\,L}{A\;E}\,\beta\left(1+\frac{1}{2}\,\beta\right)=\frac{16\,f}{\pi\,l}\left(1+\frac{\beta}{2}\right)\left(a_1+\frac{a_3}{3}+\frac{a_5}{5}+\ldots.\right)$$
$$+\frac{\pi^2}{4\,l}\,(1+\beta)\,(a_1{}^2+2^2\,a_2{}^2+3^2\,a_3{}^2+\ldots.)\ldots\ldots\ldots\ldots(18)$$

The coefficients, a_1, a_2, a_3, . . ., can be calculated in each particular case as already explained (Section IV), so that Equation (18) establishes a definite relation between β and the live load, p. The simplest way to solve this equation for any particular load distribution is to assume a certain value for β, that is, a certain increase in horizontal component of cable stress and to calculate, from Equation (18), the intensity of the live load, p, necessary to produce such an increase in cable stress. Then assuming, for small fluctuation in cable stress, that β is proportional to p, the additional cable stress for any value of p can easily be calculated.

In order to show the procedure take the numerical example already considered, and assume that, f = 145.3 ft.; A = 275 in. square = area of cross-section of cable; and w = 5 820 lb. per lin. ft. = intensity of dead load. Then, using the previous data,

$$\alpha=\frac{H_w\,l^2}{E\,I\,\pi^2}=1.75$$

and,

$$L=\int_0^l\frac{d\,s^3}{d\,x^2}=1\,564\text{ ft.}$$

Consider the case where the live load covers only one-quarter of the span to the left support of the truss and the deflection curve is represented by Equation (13). Assume that the intensity, p, of the live load is such that a definite increase in cable stress is produced. Take, for instance,

$$\beta=\frac{H}{H_w}=0.2$$

Then the series, Equation (13), becomes,

$$\eta=\frac{2\,l^4}{E\,I\,\pi^5}\left(\frac{0.2929\,p-0.4\,w}{1+1.75\times1.2}\sin\frac{\pi\,x}{l}+\frac{p\sin\dfrac{2\,\pi\,x}{l}}{2^5+2^3\times1.75\times1.2}\right.$$
$$+\frac{1.7071\,p-0.4\,w}{3^5+3^3\times1.75\times1.2}\sin\frac{3\,\pi\,x}{l}+\frac{2\,p\sin\dfrac{4\,\pi\,x}{l}}{4^5+4^3\times1.75\times1.2}$$
$$\left.+\frac{1.7071\,p-0.4\,w}{5^5+5^3\times1.75\times1.2}\sin\frac{5\,\pi\,x}{l}+\ldots.\right)\ldots\ldots\ldots\ldots\ldots(19)$$

and, hence,

$$a_1+\frac{1}{3}\,a_3+\frac{1}{5}\,a_5+\ldots=\frac{2\,l^4}{E\,I\,\pi^5}\,(0.0965\,p-62.81)$$

and,

$$a_1{}^2+2^2\,a_2{}^2+3^2\,a_3{}^2+\ldots=\left(\frac{2\,l^4}{E\,I\,\pi^5}\right)^2(0.01096\,p^2-11.89\,p+3\,920)$$

Substituting these values in Equation (18), the following quadratic equation for determining p, will be obtained,

$$5.45 = 1.827(0.0965\,p - 62.81) + 0.001797(0.01096\,p^2 - 11.89\,p + 3\,920)$$

or,

$$0.0000197\,p^2 + 0.1555\,p - 113.4 = 0$$

from which,

$$p = 671 \text{ lb. per lin. in.}$$

This is the intensity of the live load necessary to produce an increase, H, in the horizontal component equal to $0.2\,H_w$. The increase in cable stress for any other intensity of live load can easily be calculated. Assume, for instance, that a live load of 4 000 lb. per lin. ft. covers the truss between the limits, $x = 0$ and $x = \frac{l}{4}$; then the corresponding increase in the horizontal component of the cable stress will be,

$$H = \frac{0.2\,H_w \times 4\,000}{12 \times 671} = 1\,040\,000 \text{ lb.}$$

In these calculations it has been assumed that β varies in the same proportion as the load, p; but as can be seen from the fundamental Equation (18), this assumption is not strictly correct. In order to show what errors may be involved in such an assumption additional calculations were made for the load covering only one-quarter of the span to the left support of the truss and in two different ways.* In the first case, after calculating p for $\beta = 0.05$, in the same manner as before, the values of p for other values of β were found on the assumption that there is a simple proportionality between β and p. In this manner the figures in the second line of the following tabulation were obtained. In the second case, the loads, p', were calculated from Equation (18) independently for each value of β.

Value of β =	0.05	0.1	0.2	0.4
Value of p =	169.4	338.8	677.6	1 355
Value of p' =	169.4	338.5	673.0	1 322

It is seen from this tabulation that there is a slight deviation from the proportionality between β and p, namely, β is increasing somewhat more rapidly than p. This means that with the increase in load, p, a large proportional part of the load will be taken by the cable. At the same time this deviation from proportionality is small and a conclusion can be made that in calculating p from Equation (18) a rough approximation for β will be sufficient for obtaining p with a satisfactory accuracy.

In the deviation of the general expression for the work done by the load acting on the cable (see Equation (17)), the average value, $w + \frac{q}{2}$, for the load was used. This is strictly correct only when the deflections are proportional to the loads. To find what errors are involved in this method of calculation an additional investigation was made by dividing the process of loading into two steps, each step representing the loading by a load of one-half the final intensity. In calculating the cable stress for the first step, the previous method was used. For the second step the calculations of the cable stress were made by taking into account the changes in the cable stress and in the loads produced by the loadings of the first step. These calculations showed that the errors involved in this method of determining the work done by the loads acting on the cable are very small for the usual values of β and can be neglected.

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* These calculations were made by Professor G. C. Priester. A more accurate value for a, namely, 174.6, was taken in this case and all calculations were made to four significant figures.

In addition, it is interesting to note that the second term on the right side of Equation (18), representing the influence of deflection of the truss on the distribution of the load transmitted to the cable, is usually small. This fact explains why the ordinary theory of suspension bridges, which assumes that the load acting on the cable is uniformly distributed, yields satisfactory results.

VI.—THE CASE OF THREE SPANS

The method just developed can be applied in the case of three spans, such as shown in Fig. 5. Assuming that the cable can move without friction at the top of the towers, the horizontal component of the cable stress will be constant along all three spans. For calculating the deflections of the middle span the equations already derived can be used.

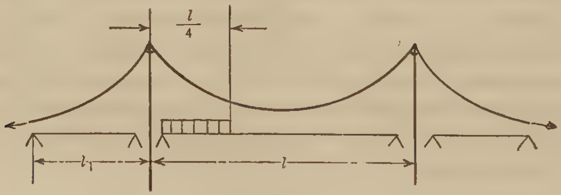

FIG. 5.

In considering the side spans let,

l_1 = length of side span;
w_1 = dead load of side span; and
EI_1 = flexural rigidity of side-span truss.

Taking the deflection curve of the side-span truss in the form of the series:

$$\eta_1 = b_1 \sin\frac{\pi x}{l_1} + b_2 \sin\frac{2\pi x}{l_1} + b_3 \sin\frac{3\pi x}{l_1} + \ldots\ldots\ldots\ldots \quad (9a)$$

in which, x is measured from the left end of the span. The coefficients, b_1, b_2, ..., can be calculated by using equations similar to Equations (12). Assuming, for instance, that there is no live load on the side spans (Fig. 5), Equation (14) becomes

$$\eta_1 = -\frac{4\beta w_1 l_1^4}{EI_1 \pi^5}\left(\frac{\sin\frac{\pi x}{l}}{1+\alpha_1(1+\beta)} + \frac{\sin\frac{3\pi x}{l}}{3^5+3^3\alpha_1(1+\beta)} + \cdots\right) \ldots (14a)$$

in which,

$$\alpha_1 = \frac{H_w \, l_1^2}{E \, I_1 \, \pi^2}$$

For calculating the additional horizontal component, H, of the cable stress an equation analogous to Equation (18) can be derived. For this purpose, it is only necessary to put the complete length of the cable for L on the left side of Equation (18) and add to the right side the work done by loads acting on the side parts of the cable. This latter work will be:

$$T_1 = 2 \int_0^{l_1} \left(w_1 + \frac{w_1 \beta}{2} - \frac{H_w}{2} (1 + \beta) \frac{d^2 \eta_1}{d x^2} \right) \eta_1 \, d x$$

Substituting the series, Equation (9a), for η_1,

$$T_1 = 2 \left(w_1 + \frac{w_1 \beta}{2} \right) \frac{2 \, l_1}{\pi} \left(b_1 + \frac{1}{3} b_3 + \frac{1}{5} b_5 + \ldots \right)$$
$$+ \frac{\pi \, 2 \, H_w}{2 \, l_1} (1 + \beta) \, (b_1^2 + 2^2 \, b_2^2 + 3^2 \, b_3^2 + \ldots.) \ldots\ldots\ldots\ldots (20)$$

The equation for determining H will be,

$$\frac{H_w^2 \, L}{A \, E} \beta \left(1 + \frac{1}{2} \beta \right) = \left(w_1 + \frac{w_1 \beta}{2} \right) \frac{4 \, l_1}{\pi} \left(b_1 + \frac{1}{3} b_3 + \frac{1}{5} b_5 \right)$$
$$+ \frac{\pi^2 \, H_w}{2 \, l_1} (1 + \beta) \, (b_1^2 + 2^2 \, b_2^2 + 3^2 \, b_3^2 + \ldots) + \left(w + \frac{w \, \beta}{2} \right) \frac{2 \, l}{\pi}$$
$$\left(a_1 + \frac{1}{3} a_3 + \frac{1}{5} a_5 + \ldots \right) + \frac{\pi^2 \, H_w}{4 \, l} (1 + \beta) \, (a_1^2 + 2^2 \, a_2^2 + 3^2 \, a_3^2 + \ldots). (21)$$

For the numerical example already considered assume that,

l_1 = 713.5 ft.
w_1 = 6 130 lb. per lin. ft.
$E \, I_1$ = 29 × 10^6 × 50 860 × 144 lb. per sq. in.; and
L = 3 452 ft.

Then, assuming the loading shown in Fig. 5, the deflection curve of the middle span truss will be given by Equation (19) as before. The deflection curves for the side span trusses will be represented by Equation (14a), so that, substituting the numerical values, and taking $\beta = 0.2$,

$$\alpha_1 = \frac{H_w \, l_1^2}{E \, I_1 \, \pi^2} = 0.3664$$

and,

$$\eta_1 = - \, 23.46 \sin \frac{\pi \, x}{l} - 0.132 \sin \frac{3 \, \pi \, x}{l} + \ldots..$$

then,

$$b_1 + \frac{1}{3} b_3 + \ldots.. = - \, 23.5 \text{ in.}; \; b_1^2 + 3^2 \, b_3^2 + \ldots = 552 \text{ in. square.}$$

Substituting in Equation (21),

$$11.97 = 1.827 \, (0.0965 \, p - 62.81) + 0.001797 \, (0.01096 \, p^2 - 11.89 \, p + 3\,920)$$
$$- \, 13.8 + 0.38$$

or,

$$0.0000197\ p^2 + 0.1555\ p - 133.3 = 0$$

from which,

$$p = 780 \text{ lb. per lin. in.}$$

If a load of the intensity of 4 000 lb. per ft. is acting on the part of the middle span shown in Fig. 5, the increase in the horizontal component will be,

$$H = \frac{0.2\ H_w \times 4\ 000}{12 \times 780} = 897\ 000 \text{ lb.} \pm$$

VII.—Effect of Change in Temperature

The same method can also be used in calculating the additional horizontal component, H, of the cable stress produced by a change in temperature. Let,

$\omega = 66 \times 10^{-7}$ = coefficient of linear expansion per degree Fahrenheit; and

$$L_t = \int_0^l \frac{d\ s}{d\ x}\ d\ s$$

Then, the work in the cable due to temperature is:*

$$\int_0^l \left(H_w + \frac{H}{2}\right) \frac{d\ s}{d\ x}\ \omega\ t\ d\ s = H_w \left(1 + \frac{\beta}{2}\right) \omega\ t\ L_t$$

Using for the change in potential energy of the cable, Equation (10), as before, the total work in the cable becomes,

$$\frac{H_w^2\ \beta \left(1 + \frac{\beta}{2}\right)}{A\ E}\ L + H_w \left(1 + \frac{\beta}{2}\right) \omega\ t\ L_t$$

This work must be equal to that done by the vertical load acting on the cable. For this later work Equations (17) and (20) will be used. The equation for determining H will be,

$$\frac{H_w^2\ \beta \left(1 + \frac{\beta}{2}\right)}{A\ E}\ L + H_w \left(1 + \frac{\beta}{2}\right) \omega\ t\ L_t = \left(w + \frac{w\ \beta}{2}\right) \frac{2\ l}{\pi}$$

$$\left(a_1 + \frac{1}{3}\ a_3 + \frac{1}{5}\ a_5 + \dots\right) + \frac{\pi^2\ H_w}{4\ l}\ (1 + \beta)\ (a_1^2 + 2^2\ a_2^2 + 3^2\ a_3^2 + \dots)$$

$$+ \left(w_1 + w_1 \frac{\beta}{2}\right) \frac{2\ l_1}{\pi} \left(b_1 + \frac{1}{3}\ b_3 + \frac{1}{5}\ b_5 + \dots\right)$$

$$+ \frac{\pi^2\ H_w}{4\ l_1}\ (1 + \beta)\ (b_1^2 + 2^2\ b_2^2 + 3^2\ b_3^2 + \dots\dots) \dots\dots\dots\dots (22)$$

Applying these equations, as before, to a bridge with three spans and assuming that $\beta = 0.2$ and $L_t = \int \frac{d s^2}{d\ x} = 3\ 324$ ft., and letting $p = 0$,

$$11.97 + 1.1\ \omega\ t \times 3\ 324.12 = -\ 1.827 \times 62.81 + 0.001979 \times 3\ 920$$
$$-\ 13.8 + 0.38$$

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* It is assumed that the fluctuations in the cable stress are small; and the average stress, $H_w + \frac{H}{2}$, is taken in calculations.

or,

$$1.1 \times 3\,324 \times 12 \times \omega\, t = -\,133.3$$

from which,

$$\omega\, t = -\,0.00304$$

Assuming that $\omega = 66 \times 10^{-7}$ in. per in. per degree Fahrenheit,

$$t = -\,461° \text{ Fahr.}$$

Now the additional horizontal component, H, for any variation in temperature can easily be calculated. Assuming, for instance, that the temperature increased 55° Fahr. above normal,

$$H = -\,\frac{H_w \times 0.2 \times 55}{461} = -\,250\,000 \text{ lb.}$$

Combining this effect of the increase in temperature with that already calculated, due to the load distributed as shown in Fig. 5, the complete change in the horizontal component of cable stress will be,

$$H = 897\,000 - 250\,000 = 647\,000 \text{ lb.}$$

which differs only about 2½% from the result given by Messrs. Johnson, Bryan, and Turneaure.* This agreement seems satisfactory considering that all the calculations were made with a slide-rule and that an approximate value for the coefficient of thermal expansion was used. A better approximation will be obtained by using the given value of $E\,I$ as a first trial and by calculating a more accurate value of β from Equation (22).

CONCLUSIONS

It has been shown that by using a trigonometric series for the deflection curve of the span truss a direct calculation of the change in the horizontal component of the cable stress becomes possible; and that the actual distribution of the live load can easily be taken into account in this calculation. Due to the quick convergence of the series representing the deflection curves, the calculation of the deflections and of the bending moments in the span truss can be made without difficulty and in a much shorter time than by the usual method.

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* "The Theory and Practice of Framed Structures," Vol. II, p. 296.

DISCUSSION

Lloyd G. Frost,* Assoc. M. Am. Soc. C. E. (by letter).—The method developed by the author should serve to advantage as a check on the established procedure; its mathematical concept is clear and ably expounded. The title, however, is misleading, giving as it does the impression that the subject matter deals with the stiffening system from the standpoint of comparative design, whereas it is found to be a purely mathematical treatise in which a method is developed by which calculations can be made in a "much shorter time than by the usual method."

This description suggests the possibility of applying this method to the preparation of cost estimates. In this connection the writer has had occasion to prepare complete and exhaustive estimates and investigations of the cost of a proposed suspension bridge over the Mississippi River at New Orleans, La. These investigations were extended to comparisons of fixed and rocker towers, eye-bar and wire cables, also the combination of the cables with a part of the top chord of the stiffening truss, and the variation of the depth of the stiffening truss after the method used by D. B. Steinman and William G. Grove, Members, Am. Soc. C. E., in the design of the Florianopolis Bridge,† was reviewed as well.

Estimates were first based on the elastic theory and then revised by the deflection method. Finally, the results obtained were compared with estimates prepared from the data and curves presented by J. A. L. Waddell, M. Am. Soc. C. E.‡ The agreement was within 3%, which amounts to a practical check.

The considerable labor involved in determining a satisfactory basis of estimate amounted virtually to a complete design as regards fundamentals, and the results finally obtained indicate that for all practical purposes of preliminary estimates, curves such as those prepared by Mr. Waddell are sufficiently accurate. It is well known that very few engineers are called upon to design suspension bridges; and then, in many cases, such work goes no further than preliminary estimates of cost.

Most engineers are inclined to view with a certain skepticism any mathematical "short-cuts" with which they are unfamiliar; and, if used, to check them thoroughly with methods they are accustomed to employ. The paper under discussion presupposes a facility with the calculus which, in the writer's opinion, a majority of able and practical designers do not have. The method developed in this paper is to some extent shorter than that ordinarily used in calculations of this nature; but the practical value of such shortening seems somewhat doubtful.

On the rare occasions that the engineer is called upon to design suspension bridgework, his first and in all probability his only necessity for quick methods of calculation will have to do with the preparation of preliminary estimates of cost. Once the work is in the hands of the designers for final development, the matter of time involved in design calculation becomes of small moment; and the calculation of bending moments and deflections in the truss constitutes a very small part of the work involved.

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* Cons. Engr., Trenton, N. J.

† *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 266.

‡ *Loc. cit.*, Vol. 91 (1927), p. 884.

While perhaps of interest to the engineer who has a flair for abstract mathematical analysis, the writer believes this paper to be largely another leaf in the already imponderable tome of matter that serves but a small minority and is of little value to the busy engineer in the practice of his profession.

Hans H. Rode,* M. Am. Soc. C. E. (by letter).—This paper, presented by an eminent authority in the field of applied mathematics, shows that trigonometric series can be utilized to good advantage in computations pertaining to suspension bridges. It is to be hoped that in the future such series will be as well known and as frequently used by civil engineers as they are now by the mechanical and electrical engineers.

The writer, however, cannot agree with the more specific procedure recommended in this paper. The author assumes that the additional horizontal component, H, from the live load, p, may be neglected in comparison with the horizontal component, H_w, from the dead load, w, and, on this basis, he finds a maximum deflection, $\eta_{\text{max.}} = 22$ in., for a concentrated load of $P = 100\,000$ lb. at the middle of the main span of the Manhattan Bridge.

This is correct enough according to the author's method, which amounts to assuming the cables carried over sheaves and counterweighted so as to maintain a constant stress in them, but it does not represent actual conditions in suspension bridges. If the assumption were correct, large bridges would deflect 30 or 40 ft., or more, and their stiffening trusses would break. Using the author's numerical data, the correct value of $\eta_{\text{max.}}$ is found to be approximately 2 in.

In computing $\eta_{\text{max.}}$ the author has entirely ignored the condition that the horizontal projection of the cable must equal the distance between anchorages. Without this condition, in some form or other, the deflections and stresses cannot be determined.

The deductions in this paper are also based on Equation (4) which is frequently encountered in books and discussions and seems to be universally accepted as correct—which it is not.

The writer has found that the exact differential equation is rather complicated, but that it assumes a simple form if the cable strain—that is, the elongation of the cable under live load stress or from temperature, is neglected. In that case, and assuming a parabolic cable curve, the equation will be:

$$E\,I\,\frac{d^4\,\eta}{d\,x^4} = p - \beta\,w + H_w\,(1+\beta)\,\frac{d}{d\,x}\left(\frac{d\,\eta}{d\,x}\sec^2\phi\right) = p - \beta\,w$$

$$+ H_w\,(1+\beta)\left[\frac{d^2\,\eta}{d\,x^2} + \frac{d^2\,\eta}{d\,x^2}\,t\,g^2\,\phi - \frac{d\,\eta}{d\,x}\,\frac{16\,f}{l^2}\,t\,g\,\phi\right] \ldots\ldots\ldots(23)$$

in which, ϕ = the slope angle of the cable curve at any point, and,

$$\sec^2\phi = 1 + t\,g^2\,\phi = a + b\,x + c\,x^2$$

Equations (4) and (23) differ only in the last term on the right-hand side.

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* Prof., Norges Tekniske Höiskole, Trondhjem, Norway.

Ordinarily, Equation (4) constitutes a good approximation. This is particularly true for cables with a small sag ratio, although the effect depends largely also on the type of loading. It should be borne in mind, however, that the errors resulting from the use of Equation (4) are, in some cases, not altogether negligible.

E. STEUERMANN,* ESQ. (by letter).—The solution of the problem presented by Professor Timoshenko is of universal significance and can be applied not only when the moment of inertia of the stiffening truss is constant within the entire span, but also when it is variable.

The author's Equation (4) may also be written in the form:

$$E\,I\frac{d^2\,\eta}{d\,x^2} - H_w\,(1+\beta)\,\eta = -M_x + \beta\,H_w\,y \ldots\ldots\ldots\ldots (24)$$

in which, M_x is the bending moment due to the action of the external load on the truss. This formula is equally effective when the value of I is variable.

Suppose, for example, that I_1, the moment of inertia of the truss at the ends, is less than I_0, the moment of inertia at the center. This is quite a rational assumption if the truss is not continuous through the towers. The value of I may be found by the equation:

$$I = I_0 \sin\left(\frac{m\,\pi\,x}{l} + \alpha\right) \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

in which, $\alpha = \sin^{-1}\left(\frac{I_1}{I_0}\right)$ and $m = 1 - \frac{2\,\alpha}{\pi}$. When $I_1 = \frac{I_0}{2}$, then $\alpha = \frac{\pi}{6}$ and $m = \frac{2}{3}$.

Equation (25) imposes a definite and, in many cases, an admissible restriction on the manner in which the moment of inertia, I, shall vary, but is considered only for a specific case. The method of calculation that follows is equally applicable for the general case:

$$I = I_0\,f\,(x) \ldots\ldots\ldots\ldots\ldots\ldots (26)$$

Expanding the right side of Equation (24) into the series of Fourier:

$$-M_x + \beta\,H_w\,y = -\left(b_1 \sin\frac{\pi\,x}{l} + b_2 \sin\frac{2\,\pi\,x}{l} + \ldots + b_n \sin\frac{n\,\pi\,x}{l}\right) \ldots (27)$$

in which,

$$b_n = \frac{2}{l}\int_0^l (M_x - \beta\,H_w\,y)\,d\,x \ldots\ldots\ldots\ldots\ldots (28)$$

The integral, η, may be presented in the form of a series, as follows:

$$\eta = a_1 \sin\frac{\pi\,x}{l} + a_2 \sin\frac{2\,\pi\,x}{l} + \ldots + a_n \sin\frac{n\,\pi\,x}{l} + \ldots = \sum_1^\infty a_n \sin\frac{n\,\pi\,x}{l} \ldots (29)$$

Substituting in Equation (24), the values found by Equations (27) and (29), and changing signs:

$$\frac{\pi^2\,E\,I_0}{l^2} \sin\left(\frac{m\,\pi\,x}{l} + \alpha\right) \sum_1^\infty n^2\,a_n \sin\frac{n\,\pi\,x}{l} + H_w\,(1+\beta) \sum_1^\infty a_n \sin\frac{n\,\pi\,x}{l}$$
$$= \sum_1^\infty b_n \sin\frac{n\,\pi\,x}{l} \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

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* Prof., Kief Polytechnic High School, Kief, Union of Socialistic Soviet Republics.

To determine the coefficients, a_n, multiply both sides of Equation (30) by $\sin\frac{\kappa\pi x}{l}$ (in which, $\kappa = 1, 2, 3 \ldots n$), and integrate between the limits 0 and l, then,

$$\frac{\pi^2 E I_0}{l^2}\sum_1^\infty n^2 a_n \int_0^l \sin\left(\frac{m\pi x}{l}+\alpha\right)\sin\frac{m\pi x}{l}\sin\frac{\kappa\pi x}{l}\,dx + H_w(1+\beta)\sum_1^\infty a_n \int_0^l \sin\frac{n\pi x}{l}\sin\frac{\kappa\pi x}{l}\,dx = \sum_1^\infty b_n \int_0^l \sin\frac{n\pi x}{l}\sin\frac{\kappa\pi x}{l}\,dx \qquad (31)$$

By the known formulas of integral calculus:

$$\int_0^l \sin\left(\frac{m\pi x}{l}+\alpha\right)\sin\frac{n\pi x}{l}\sin\frac{\kappa\pi x}{l}\,dx = \frac{2\left[(-1)^{n+\kappa+1}-1\right] m\, l\, n\,\kappa}{\pi\left[(n-\kappa)^2-m^2\right]\left[(n+\kappa)^2-m^2\right]} \qquad (32)$$

Substituting Equation (32) in Equation (31) and transposing the terms:

$$a_\kappa + \frac{4\pi E I_0 m\cos\alpha}{H_w l^2(1+\beta)}\sum_1^\infty \frac{\left[(-1)^{n+\kappa+1}-1\right]\kappa n^3 a_n}{\left[(n-\kappa)^2-m^2\right]\left[(n+\kappa)^2-m^2\right]} = \frac{b_\kappa}{H_w(1+\beta)} \qquad (33)^*$$

in which, $\kappa = 1, 2, 3, \ldots . n$. Let,

$$\mu = \frac{4\pi E I_0 m\cos\alpha}{H_u l^2(1+\beta)} \qquad (34)$$

$$N_{\kappa n} = \frac{\left[(-1)^{n+\kappa+1}-1\right]\kappa n^3}{\left[(n+\kappa)^2-m^2\right]\left[(n-\kappa)^2-m^2\right]} \qquad (35)$$

and,

$$C_\kappa = \frac{b_\kappa}{H_w(1+\beta)} \qquad (36)$$

Then, Equation (33) becomes:

$$a_\kappa + \mu\sum_1^\infty N_{\kappa n} a_n = C_\kappa \qquad (37)$$

If Equation (29) is restricted to a definite number of terms, that is, γ-terms, or enough to express the value of η accurately, Equation (37) will also include the same definite number of terms, such that $\kappa = 1, 2, 3, \ldots . \gamma$.

Solving Equation (37) within these definite limits will express a_1, a_2, a_3 $\ldots$ a_γ, as well as η as functions of β. The author derives Equation (18) for use in determining the value of β and the same equation can be used in the present case. The remainder of the solution follows exactly the method described by the author for trusses of constant section.

The solution of Equation (37) is simplified due to the fact that it may be separated into two independent parts, one containing the term, a_κ, for even values of κ and the other, for odd values of κ. This is made evident by the fact that $N_{\kappa n} = 0$, when κ is even and n is odd, or *vice versa*.

If the value of I is determined by Equation (26), the integrals,

$$\int_0^l f(x)\sin\frac{n\pi x}{l}\sin\frac{\kappa\pi x}{l}\,dx$$

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* When $\alpha = 0$, $m = 1$, and $n = \kappa = \pm 1$, the right side of Equation (32) assumes the indeterminate form, $\frac{0}{0}$, the limiting value of which is 0.

(in which, κ and $n = 1, 2, 3, \ldots n$), would appear in Equation (33). In many cases, for instance, when $f(x)$ is a polynomial or an exponential function, it is possible to solve them exactly, but generally they can be determined easily by some approximate method.

The theory herein presented may be applied to the numerical example given by the author, except that the moment of inertia diminishes, according to Equation (25), to one-half the given value at the support.

First, compute the coefficients, b_n. In the given case it will be easier to use, not Equation (28), but the known equation:

$$\frac{d^2 M}{d x^2} = - q \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

in which, M is the bending moment of the truss, and q is the load. In this specific example, $q = p - \beta w$ when $0 \leq x \leq \frac{l}{4}$ and $q = - \beta w$ when $\frac{l}{4} \leq x \leq l$.

Expanding q into a trigonometric series and integrating twice (see Equation (38)):

$$M_x - \beta H_w y = \sum_1^\infty \frac{2 l^2 \left\{ p \left(1 - \cos \frac{n \pi}{4}\right) - \beta w \left[1 - (-1)^n\right] \right\} \left(\sin \frac{n \pi x}{l}\right)}{\pi^3 n^3} \ldots (39)$$

Then, from Equation (36):

$$C_n = \frac{2 l^2 \left\{ p \left(1 - \cos \frac{n \pi}{4}\right) - \beta w \left[1 - (-1)^n\right] \right\}}{H_w (1 + \beta) \pi^3 n^3} \ldots\ldots\ldots (40)$$

The author has demonstrated that, in order to determine η, it will be sufficient to restrict the expansion of Equation (29) to four terms so that in Equation (37), the value of γ is 4. It is almost evident from the mechanical calculations that the deflection curve can be represented in the form of four sinusoids. Using the data suggested by the author, Equation (37) becomes:

$$\left.\begin{aligned} a_1 + \mu \left(\frac{81}{64} a_1 - \frac{27.81}{16.140} a_3\right) &= C_1 \\ a_2 + \mu \left(- \frac{162}{35} a_2 - \frac{81}{40} a_4\right) &= C_2 \\ a_3 + \mu \left(- \frac{243}{16.140} a_1 + \frac{81^2}{640} a_3\right) &= C_3 \\ a_4 + \mu \left(- \frac{81}{100} + \frac{32.81}{143} a_4\right) &= C_4 \end{aligned}\right\} \ldots\ldots\ldots\ldots\ldots (41)$$

From this point the solution is not difficult and, therefore, does not need to be presented. Its application to the particular case presented by the author, namely, to find the corresponding values of p when $\beta = 0.2$, is as follows: Calculating the value of p from Equation (34), the values of a in

Equation (41) can be easily found in terms of C, by making the proper substitutions and solving simultaneous equations. The results are:

$$\left.\begin{aligned} a_1 &= 0.6944\ C_1 + 0.0517\ C_3 \\ a_2 &= 0.3842\ C_2 + 0.0371\ C_4 \\ a_3 &= 0.0057\ C_1 + 0.2184\ C_3 \\ a_4 &= 0.0093\ C_2 + 0.1370\ C_4 \end{aligned}\right\} \dots\dots\dots\dots (42)$$

In order to make the remaining calculations shorter, it is best to introduce the quantities:

$$\kappa = \frac{2\ l^2}{H_w\ (1+\beta)\ \pi^3} = 21.99 \dots\dots\dots\dots (43)$$

$$C_n = \kappa\ C_n{}' \dots\dots\dots\dots (44)$$

$$a_n = \kappa\ a_n{}' \dots\dots\dots\dots (45)$$

Then, by making proper substitutions in Equation (40) values of C are determined as follows:

$$\left.\begin{aligned} C_1{}' &= \frac{0.2929\ p - 2\ \beta\ w}{1} \\ C_2{}' &= \frac{p}{8} \\ C_3{}' &= \frac{1.7071\ p - 2\ \beta\ w}{27} \\ C_4{}' &= \frac{p}{32} \end{aligned}\right\} \dots\dots\dots\dots (46)$$

Consequently, according to Equation (42):

$$\left.\begin{aligned} a_1{}' &= 0.2067\ p - 1\ 621 \\ a_2{}' &= 0.0492\ p \\ a_3{}' &= 0.0155\ p - 32.126 \\ a_4{}' &= 0.0054\ p \end{aligned}\right\} \dots\dots\dots\dots (47)$$

In the author's Equation (18), substitute the values of a_n found by Equations (45) and (47) and also the other numerical values given; then the value of p will be found equal to 7 937 lb. per lin. ft. = 661.5 lb. per lin. in. Beyond this point the solution is exactly as given by the author.

As a second example, consider the same bridge, assuming that $I_1 = 0$ and that $\beta = 0.2$, as before. In Equation (25), since $\alpha = 0$ and $m = 1$:

$$I = I_0 \sin \frac{\pi\ x}{l} \dots\dots\dots\dots (48)$$

Then, following the same procedure as in Equation (46):

$$\left.\begin{aligned} C_1 &= 1.404\ a_1 - 0.727\ a_3 \\ C_2 &= 2.294\ a_2 - 1.479\ a_4 \\ C_3 &= -\ 0.081\ a_1 + 3.850\ a_3 \\ C_4 &= -\ 0.370\ a_2 + 5.930\ a_4 \end{aligned}\right\} \dots\dots\dots\dots (49)$$

and,

$$\left.\begin{aligned} a_1{}' &= 0.221\ p - 1\ 692 \\ a_2{}' &= 0.0653\ p \\ a_3{}' &= 0.021\ p - 58.7 \\ a_4{}' &= 0.0098\ p \end{aligned}\right\} \dots\dots\dots\dots (50)$$

The author's Equation (18) will then become:

$$0.266 \times 10^{-5}\, p^2 + 0.199\, p - 1\,689.4 = 0$$

$$p = 7\,700 \text{ lb. per lin. ft.} = 642 \text{ lb. per lin. in.}$$

Comparing these values of p (661.5 and 642) with those determined by the author for a truss with a constant rigidity ($p = 671$ lb. per lin. in.), will show that the magnitude of the load necessary to cause a definite increase, H, in the stress, H_w, will diminish very slowly with a decrease in the rigidity of the truss. When the moment at the support is reduced one-half, the value of p decreases only 9.5 lb. per lin. in., whereas when the moment at the support is zero, the decrease is 29 lb. per lin. in. (A somewhat greater decrease would be obtained if a less favorable curve than that described by Equation (25) were assumed.) In other words, the horizontal component as well as the total tension in the cable increases under the action of a given temporary load at a rate considerably slower than the corresponding decrease in the rigidity of the truss. This result may interest the bridge engineer, because it will, perhaps, allow a more rational construction of the stiffening truss.

George C. Priester,* Esq. (by letter).—The application of a trigonometric series to a deflection problem is so advantageously presented in this paper that it is desirable to discuss the problem of the stiffened suspension bridge in more detail.

The writer presents an extension of this problem in the following studies: (*a*) The relation between the additional horizontal component of cable stress and live load; (*b*) the application of the live load in two increments; (*c*) the solution of a problem when the cable is attached to the towers; and (*d*) the effect of a concentrated load on the span. The notations used are the same as those in the paper.

The Relation Between β and p.—Since p represents the live load per unit length over a part or all of the truss, and $\beta = \frac{H}{H_w}$, the relation between these two values will be used in terms of numerical quantities. For this problem the relation between p and β will be investigated for a three-span suspension bridge with the cable free to move over the towers. It will be assumed that the live load covers the main span from $x = 0$ to $x = \frac{l}{4}$; that there is no live load on the side spans; and that the temperature is 55° Fahr. above normal. The other numerical data for this problem are those of the Manhattan Bridge as given by the author.

In order to solve the numerical computations it is desirable to assume values for β in Equation (22) and compute p. This will be done in various ways: First, by assuming the load and temperature to act simultaneously, that is, by using the complete equation; second, by assuming the same values for β and the temperature to be normal; and third and fourth, to carry out the same computations by omitting the terms involving the square of the coefficients, a_n and b_n, for the first and second case. The values of p obtained by the first and second method will be called p (exact) in contrast with those obtained by the third and fourth method in which they will be called p (approximate). The p (approximate) values correspond to those obtained by the deflection method, since the original assumptions in the development of the equations for these two cases are identical. The results for these conditions are given in Table 1.

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TABLE 1.—VARIATION OF p AND β. WITH AND WITHOUT TEMPERATURE (THREE SPANS).

β.	p (exact) $t = 55°$ Fahr.	p (exact) without allowing for temperature.	Difference.	p (approximate) $t = 55°$ Fahr.	p (approximate) without allowing for temperature.	Difference.
0.05	277.1	196.1	81.0	280.0	197.2	82.8
0.10	474.3	391.8	82.5	481.2	395.7	85.8
0.20	866.5	781.5	85.0	887.5	797.0	90.5
0.40	1 641.0	1 552.0	89.0	1 717.0	1 616.0	101.0

It will be noted that the values in the "Difference" columns increase with an increase in β. This is to be expected since the term in Equation (22) which takes into account the effect of temperature is of the form,

$$H_w \left(1 + \frac{\beta}{2}\right) \omega \, t \, L_t$$

and an increase in β will increase the effect of a given temperature.

It should be noted that there is a slight deviation from a linear proportionality between p and β. Since this deviation is small it is possible to assume a linear relation for small increments of β.

The Live Load Applied in Two Increments.—In the development of Equation (22) the average load, $w + \frac{q}{2}$, was assumed to be acting through the distance, η, in calculating the work done. A numerical example will show that this assumption may be used. Since a simple problem will suffice, it will be assumed that the suspension bridge consists of only one span and that the cables are fastened rigidly at the towers. The data of the main span of the Manhattan Bridge will be used. For numerical computation two-increment loading is chosen. The live load, q, acting on the cable will be divided into two parts. In the first increment the average load acting on the cable will be $w + \frac{q_1}{2}$, and, in the second increment, $w + q_1 + \frac{q_2}{2}$. The deflection of the truss for the first increment of loading will be designated by η_1 and that for the second increment by η_2. Also, let H_1 equal the additional horizontal component of cable stress due to the first increment; and H_2, that for the second increment of loading. Then, $\beta_1 = \frac{H_1}{H_w}$ and $\beta_2 = \frac{H_2}{H_w + H_1}$.

For the first increment of loading the equation for determining H_1 is:*

$$\frac{H_w^2\,L}{A\,E}\,\beta_1\left(1+\frac{\beta_1}{2}\right)=\frac{2\,w}{\pi}\left(1+\frac{\beta_1}{2}\right)\left(a_1+\frac{1}{3}\,a_3+\frac{1}{5}\,a_5+\ldots\ldots\right)$$
$$+\frac{\pi^2\,H_w}{4\,l}\,(1+\beta_1)\,(a_1^2+2^2\,a_2^2+3^2\,a_3^2+\ldots\ldots\ldots)\ldots\ldots\ldots(51)$$

and for the second increment of loading, the equation for determining H_2 is:

$$\frac{H_w^2\,(1+\beta_1^2)\,\beta_2\left(1+\frac{\beta_2}{2}\right)L}{A\,E}$$
$$=\frac{2\,w\,l}{\pi}\,(1+\beta_1)\left(1+\frac{\beta_2}{2}\right)\left(a_1'+\frac{1}{3}\,a_3'+\frac{1}{5}\,a_5'+\ldots\right)$$
$$+\frac{\pi^2\,H_w}{4\,l}\,(1+\beta_1)\,(3+2\,\beta_2)\left((a_1')^2+2^2\,(a_2')^2+3^2\,(a_3')^2+\ldots\right)\ldots(52)$$

in which,

$$a_n'=\frac{2\,l^4}{E\,I\,\pi^5}\;\frac{p\left(\cos\frac{n\,\pi\,a}{l}-\cos\frac{n\,\pi\,b}{l}\right)-\beta_2\,(1+\beta_1)\,w\,(1-\cos n\,\pi)}{n^5+n^3\,\alpha\,(1+\beta_1)\,(1+2\,\beta_2)}\ldots(53)$$

For $\beta_1 = 0.10$, the value of p from Equation (51) is 338.2 lb. per lin. in. and, for $\beta_2 = 0.10$, the value of p from Equation (52) is 368.0 lb. per lin. in. The load which will produce an additional horizontal component of cable stress corresponding to $\beta_1 + \beta_2$ will be the sum of p_1 and p_2, or 706.2 lb. per lin. in.

Since it is necessary to determine whether a single loading will produce the same effect as the two-increment loading, a single value of β (= 0.21) must be used, which is equivalent to $\beta_1 + \beta_2$. Then (Equation (51)), the value of p is 706.0 lb. per lin. in. This agrees with the value of the sum of p_1 and p_2 when the load is applied in two increments. For a second illustration of this problem assume that $\beta_1 = \beta_2 = 0.20$. When $\beta_1 = 0.20$, the value of p_1 is found to be 673.0 lb. per lin. in. by the use of Equation (51). With $\beta_2 = 0.20$ for the second increment, the value of p_2 is found to be 790.0 lb. per lin. in., by the use of Equation (52). The sum of p_1 and p_2 is 1 463.0 lb. per lin. in. The value, $\beta = 0.44$, corresponds to $\beta_1 + \beta_2 = 0.40$. From Equation (51) for $\beta = 0.44$, p is 1 461.0 lb. per lin. in. Thus, it is shown that by the use of the average load, $w + \frac{q}{2}$, a sufficiently accurate value of the cable stress is found.

An Analysis of Stress with the Cable Fixed at the Towers.—When the cable for a three-span suspension bridge is fixed at the towers, as in the case of the Delaware River Bridge, the deflection of the towers must be taken into account when a live load or a temperature change is acting on the structure. The equations necessary for the computation of the value of H for a symmetrically placed load and equal side span will be given. (Similar equations can be developed when these conditions are not symmetrical.)

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* The development of the equations used in this discussion and a more detailed study of the suspension bridge are published in "Application of Trigonometric Series to Cable Stress Analysis in Suspension Bridges,"," by George G. Priester, M. Am. Soc. C. E., *Engineering Research Bulletin No. 12,* March, 1929, Univ. of Michigan, Ann Arbor, Mich.

Since the towers are bent by a change in the cable stress and since the stress in the cable over the side span is different from that in the main-span cable, it is necessary to have an equation for the side span, one for the main span, and one for the deflection of the towers.

Besides the notations previously given, it is necessary to use the following:

H_1 = the additional horizontal component of stress in the cable of the side spans.

δ = the deflection of the towers.

I_t = the moment of inertia of a cross-section of the tower about the axis perpendicular to the plane of bending.

h = the height of the tower.

S = the vertical component of the total cable stress.

$$\beta_1 = \frac{H_1}{H_w}$$

$$L_{1s} = \int_0^{l_1} \left(\frac{d\,s_1}{d\,x}\right)^2 d\,s_1$$

$$L_{1t} = \int_0^{l_1} \left(\frac{d\,s_1}{d\,x}\right) d\,s_1$$

The equation of equilibrium for the side span is:

$$\begin{aligned} \frac{H_w^2}{A\ E}\left(1+\frac{\beta_1}{2}\right)\beta_1\,L_{1s} + H_w\left(1+\frac{\beta_1}{2}\right)(\omega\,t\,L_{1t} - \delta) \\ = \frac{2\,w_1\,l_1}{\pi}\left(1+\frac{\beta_1}{2}\right)\left(b_1 + \frac{1}{3}\,b_3 + \frac{1}{5}\,b_5 + \ldots\right) \\ + \frac{\pi^2\,H_w}{2\,l_1}(1+\beta_1)(b_1^2 + 2^2\,b_2^2 + 3^2\,b_3^2 + \,..) \ldots\ldots\ldots\ldots(54) \end{aligned}$$

and for the main span:

$$\begin{aligned} \frac{H_w^2}{A\ E}\left(1+\frac{\beta}{2}\right)\beta\,L + H_w\left(1+\frac{\beta}{2}\right)(\omega\,t\,L + 2\,\delta) \\ = \frac{2\,w\,l}{\pi}\left(1+\frac{\beta}{2}\right)\left(a_1 + \frac{1}{3}\,a_3 + \frac{1}{5}\,a_5 + \,..\right) \\ + \frac{\pi^2\,H_w}{2\,l}(1+\beta)(a_1^2 + 2^2\,a_2^2 + 3^2\,a_3^2 + \,..) \ldots\ldots\ldots\ldots(55) \end{aligned}$$

The equation for the deflection of the towers is:

$$\delta = \frac{H - H_1}{E}\int_0^h \frac{y^2}{I_t}\,d\,y \ldots\ldots\ldots\ldots\ldots\ldots(56)$$

As an example, the data taken from the Final Report of the Board of Engineers to the Delaware River Bridge Joint Commission will be used to determine the values of H, H_1, and δ. It will be assumed that a live load of 12 000 lb. per lin. ft. extends over the entire length of the main span and that there is no load on the side spans. The temperature is assumed to be 55° Fahr. above normal.

Substituting the proper numerical data in Equations (54), (55), and (56): $H = 9\,588\,000$ lb., $H_1 = 9\,391\,000$ lb., and $\delta = 19.90$ in. The value of

H which corresponds to the load of 12 000 lb. per lin. ft. is given in the aforementioned report as 9 620 000 lb. and the total movement of the tower as 20.04 in. It is evident, therefore, that the trigonometric-series method can also be adapted for computing the horizontal component of cable stress when the cable is attached at the top of the towers.

A Concentrated Load on the Span.—To simplify the calculation of the stresses in the truss of a suspension bridge it is desirable to set up an equation which gives the effect of a concentrated load on the additional horizontal component of cable stress and construct the influence diagrams.

The equation for determining the additional horizontal component of cable stress produced by a concentrated load on the truss is:

$$\frac{H_w^2 L}{A E} \beta \left(1 + \frac{\beta}{2}\right) + H_w \left(1 + \frac{\beta}{2}\right) \omega t L_t$$
$$= \frac{2 w l}{\pi} \left(1 + \frac{\beta}{2}\right) \left(a_1 + \frac{1}{3} a_3 + \frac{1}{5} a_5 + \cdots\right)$$
$$+ \frac{\pi^2 H_w}{4 l} (1 + \beta) (a_1^2 + 2^2 a_2^2 + 3^2 a_3^2 + \cdots) \ldots\ldots\ldots\ldots (57)$$

For this case,

$$a_n = \frac{2 l^4}{E I \pi^5} \; \frac{\frac{n \pi}{l} P \sin \frac{n \pi c}{l} - 2 \beta w (1 - \cos n \pi)}{n^5 + n^3 \alpha (1 + \beta)}$$

and the concentrated load, P, is at a distance, c, from the left end of the span.

As an example, Equation (57) will be applied to the calculation of the additional horizontal component of cable stress for the data of the Manhattan Bridge, with the main span regarded as a single span. The concentrated load, P, will be taken as 100 000 lb. and the temperature as normal ($t = 0$). The position of P will be assumed at various points of the span so that its effect when located at any point in the span may be determined from a diagram. The values of H for various locations of P are given in Fig. 6. With the aid of this diagram, the value of H may be determined approximately for any uniformly distributed load and the influence lines for the bending moment in the truss may be drawn.

As an illustration, it will be assumed that the uniform load extends over the left quarter of the span. It is only necessary to regard the concentrated load, P, as a unit load and then the area bounded by the curve for H, the x-axis, and the ordinates limiting the distribution of the uniform load will equal the component of cable stress for a uniformly distributed load of unit intensity. For an approximation of the area under the curve (Fig. 6) $O\ B\ C$ will be regarded as a triangle and $B\ C\ D\ E$ as a trapezoid. The combined area will equal:

$$K = \frac{0.732 \times 180.9}{2} + \frac{(1.335 + 0.732) \times 180.9}{2} = 253.1$$

For a uniform load of 4 000 lb. per lin. ft., the value of the additional horizontal component of cable stress will be, $H = 253.1 \times 4\,000 = 1\,012\,000$ lb. For similar conditions the value of H as computed by Equation (51) is 1 032 000 lb.

The error involved in using this simple approximate method is less than 2 per cent. Thus, it is evident that it may be used to determine a good approximation of the cable stress, H, or the preliminary values of β when the exact method is used for calculating H.

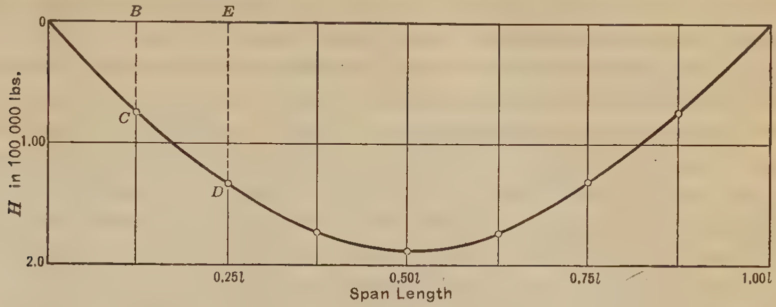

FIG. 6.

The trigonometric-series method can be used to construct the influence lines and diagrams for the bending moment in the truss. If the bending moment is written as follows:*

$$M = \left(\frac{M'}{y} - H\right) y \quad \text{(58)}$$

in which, y is the ordinate of the cable when the origin is taken at the top of the tower, then the influence line for $\frac{M'}{y}$ may be drawn in the same manner as the influence line is drawn for a simple beam (see Fig. 7). Also, from Fig. 6, the influence line for H can be drawn. The area between these two lines represents the influence diagram from which the bending moment due to a uniform load extending over any part of the span may be obtained. The shaded area in Fig. 7 represents the influence diagram when the unit load is at the point, G. This method can be used for determining the most unfavorable positions of the live load in calculating the stresses in the truss.

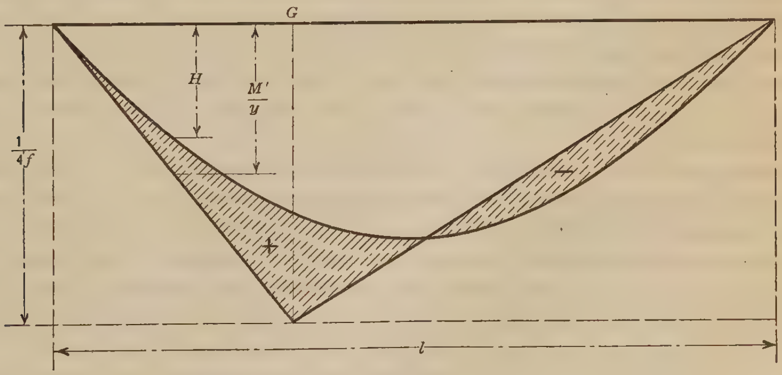

FIG. 7.

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* "Modern Framed Structures," Johnson, Bryan, and Turneaure, Ninth Edition, Pt. II, p. 221.

S. Timoshenko,[*] Esq. (by letter).—The advantages of applying trigonometric series to the investigation of cable stresses in suspension bridges were confirmed in several papers published during 1928. Martin[†] proposed this method independently at about the same time as the writer, and used it in discussing the paper by Leon S. Moisseiff, M. Am. Soc. C. E., on the Delaware Bridge.[‡] Professor Steuermann showed that the method can be applied not only when the moment of inertia of the stiffening truss is constant within the entire span, but also when it is variable. F. Karpinski[§] obtained very satisfactory results for the bending moments in the stiffening truss by using the writer's trigonometric-series method for the deflection of the truss, together with the approximate formulas of J. Resal[‖] and G. Pigeaud[¶] for the calculation of the change in cable stress. Professor Priester[**] developed the application of the trigonometric-series method for cases in which the cable is attached to the towers and a concentrated load acts on the span. On the basis of this work, he concludes that the numerical calculations in the "trigonometric-series method" are simpler than those in the "deflection method."

In discussing the paper, Mr. Frost concludes that the practical value of the proposed method as a means of lessening the work is somewhat doubtful since it "is of little value to the busy engineer in the practice of his profession." The writer must apologize if the title of the paper seems misleading, or suggests the advisability of applying his method to the preparation of cost estimates. The intention was to develop a method of analyzing cable stresses in suspension bridges without involving some of the arbitrary assumptions of the usual method, and, at the same time, introducing one that would be simple enough for practical application. The fact that a more rigorous solution is obtained with less work, justifies the application of series from the engineering point of view. The writer agrees with Mr. Frost that the paper is of the kind that serves only a small minority, and that it is of little value to the busy engineer. However, from time to time, it becomes important to discuss the fundamental assumptions upon which the usual methods of engineering analyses are based, to be sure that they are accurate enough and simple enough.

Professor Rode, referring to the first numerical example of the paper, raises the question as to the change in the cable stress due to the live load. This change can be calculated by the method developed in the paper. It can be shown in this case that it is small, and can be neglected in calculating the approximate value of the deflection. Referring to Equation (23), it is regrettable that the complete derivation was not given, in order to compare calculations made on the basis of the usually accepted Equation (4) with those made from Professor Rode's more accurate equation.

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[†] *Engineering,* Vol. 123 (1927), p. 506; see, also, *Engineering,* Vol. 125 (1928), p. 1.

[‡] *Journal,* Franklin Inst., October, 1925.

[§] *Annales* des Travaux Publics de Belgique, Juin, 1928.

[‖] "Cours de Ponts Metalliques," Tome II, Premier fascicule, 1912.

[¶] *Le Génie Civil,* 2 juillet, 1927.

[**] "Application of Trigonometric Series to Cable Stress Analysis in Suspension Bridges," by George C. Priester, M. Am. Soc. C. E., *Engineering Research Bulletin No. 12,* March, 1929, Univ. of Michigan, Ann Arbor, Mich.

The discussion of Professor Steuermann represents a very interesting development of the proposed Fourier series method. It is shown by using this method that the case of a truss with variable rigidity can easily be analyzed. In this way Professor Steuermann arrives at the interesting conclusion that a considerable reduction in rigidity at the supports of a truss has only slight effect on the magnitude of the cable stress. This result is of practical importance, and suggests the advantage of designing stiffening trusses of variable rigidity.

The discussion by Professor Priester represents several interesting examples of the application of the proposed method. It contains also calculations which prove the accuracy of this method in determining the cable stress. The discussion of the live load applied in two increments establishes the error which may be expected in applying the principle of superposition in discussing such structures as suspension bridges. The stress analysis with the cable fixed at the towers gives a simple solution in the form of a series to a problem which was formerly solved only by the tedious "cut and try" method. By using the series method, Professor Priester also obtained the influence lines for the cable stress and for the bending moments. These permit the determination of the most unfavorable loading for each cross-section of the truss.

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Paper No. 1733

THE ISTHMIAN CANAL SITUATION*

By Hans Kramer,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. William B. Landreth, Eduardo Icaza, and Hans Kramer.

Synopsis

General interest in needs for additional Isthmian canal facilities has been actively renewed. The factors which limit the capacity of the Panama Canal and the criteria upon which a forecast of the probable traffic increase is made, form the bases for a graphical method for predicting the saturation point of the existing installation. The conclusion is reached that the Panama Canal (augmented by the pending Alhajuela Reservoir development) will be adequate to handle all probable traffic until 1970.

The choice for additional facilities to be provided to meet future traffic needs is limited to the construction of additional locks at Panama and the opening of a new canal route *via* Nicaragua. The relative advantages and disadvantages of these alternatives are discussed from the present-day viewpoint. From this comparison, the installation of more locks at Panama is the logical method for supplying future needs for additional Isthmian canal facilities.

Neither the necessity nor advisability of a Nicaragua Canal appears to exist at present.

Newspapers, popular periodicals, and technical journals have contained numerous references in recent months to the impending inadequacy of the

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* This paper was written in May, 1927, and, therefore, is based on data obtainable at that time. Inasmuch as this study presents, in part, a prediction as of May, 1927, it has not been deemed desirable to modify the original manuscript by including developments of a later date. Published in August, 1928, *Proceedings*.

† Lieut., Corps of Engrs., U. S. A., Fort du Pont, Del.

Panama Canal and to the necessity of additional Isthmian canal facilities, the latter preferably by way of Nicaragua. The agitation by feature writers and editors has been augmented by the public utterances and recommendations of Congressmen and public officials.

In view of the sudden revival of public interest in a problem which has enjoyed a well-earned rest for more than a decade, it seems pertinent to raise several fundamental questions on which the present and any future discussion properly ought to be based. First, when will the maximum capacity of the Panama Canal probably be reached? Second, what additional facilities will then be advisable? In general, the various recent statements have been predicated on the hypothesis that the capacity of the Panama Canal has already been reached, or that it will be reached in the very near future, and that, therefore, the second question of additional canal facilities is of immediate importance.

It is the purpose of this paper to develop the answers to these basic questions utilizing cold-blooded statistics, established facts, and the opinions of recognized experts. Except where necessary for this purpose, no descriptive matter will be included; the written works of the builders of the Panama Canal have incomparably covered this aspect. A general familiarity with the existing canal is, therefore, assumed.

Forecast—Time of Reaching Capacity

When will the maximum capacity of the Panama Canal probably be reached?

Varying arguments have been introduced to show the necessity for additional Isthmian canal facilities. Some of these have been directly stated, others only implied. All the pertinent points, however, can be readily classified. It seems sufficient to group these arguments in two main categories—technical reasons and non-technical reasons. The latter designation is used for convenience and is not intended to be derogatory. In fact, some of these non-technical considerations may be of greater importance than the technical ones.

Technical and Non-Technical Arguments.—The technical arguments that should receive primary attention are those that portend an early inadequacy of the present Panama Canal to handle the traffic offered. Such inadequacy would be evidenced by congestion due to excessive quantity of traffic or by inability to handle vessels of excessive dimensions.

The leading argument in the non-technical group is linked with the question of National defense. The American fleet has been taken expeditiously through the Panama Canal several times. A similar canal at Nicaragua, nearer Mexico and nearer the United States, with new naval bases as an integral part of the development, would undoubtedly add to the efficacy of these naval forces. But is this necessary? The opening of new Central American markets and outlets for Central American products by way of a Nicaraguan Canal is another non-technical argument (emanating undoubtedly from political, business, and banking interests), for early commencement of canal construction in Nicaragua. Out of this argument grows another—that such an enterprise would stimulate business through the circulation of a large part of the surplus of the United States Treasury. The proponents of this particular argument may also have in mind the utilization of the Treasury's selfsame surplus to meet deficits during the long lean years before another canal becomes self-sustaining.

Some statesmen are probably hoping to make political capital out of their alertness and sagacity in being among the first to urge the construction of another or a bigger Isthmian canal. Others—perhaps more sincere, more altruistic, more philanthropic—see in a new canal *via* Nicaragua, not only the solution of the present vexing Nicaraguan question, but the settlement of the whole Central American political and economic problem for all time. All these are grouped as non-technical reasons to show the necessity for additional canal facilities.

Present Operating Capacity.—In approaching the basic question as to just when the capacity of the existing canal will be reached, it is relevant to ask the preliminary question: What is the capacity of the present canal and how is this capacity determined and limited? Concise and authoritative data on this question are given* by Brig. Gen. M. L. Walker, U. S. A., Governor of the Panama Canal, which statements form the basis of the present conclusions, and, therefore, require no further proof for their authenticity and reliability.

The limiting factors at Panama are the operating speed of the locks, their dimensions, and the water supply for maintaining the level of the summit lake. Without going into the details of mathematical computations of speed of lock operation, suffice it to say that the original designers conservatively estimated† that the Panama Canal would be able to handle 80 000 000 net tons of shipping per year. This estimate contemplated 24-hour per day operation, allowing ample shut-downs for periodic repair and overhaul. Actual experience has borne out the correctness of this estimated maximum tonnage capacity.

Each lock at Panama has usable clearances of 1 000 ft. of length, 110 ft. of width, and 40 ft. of depth. For commercial purposes these dimensions are ample. The general tendency in the design of commercial vessels shows a departure from the *Leviathan* type for reasons that are not essential to this discussion. The Washington Conference placed a "damper" on competitive naval armament and the consequent development of super-dreadnought sizes. Had the pre-war rate of growth of battleship dimensions continued without abatement the limiting size of the Panama locks would soon become a serious consideration. Therefore, the construction of both commercial and naval ships of such a size as would prevent their transit appears at present to be only a remote possibility.

Water Supply as a Limiting Factor.—The other physical factor affecting the capacity of the Panama Canal is the water supply for maintaining the level of Lake Gatun at a height to afford ample depth of channel through the summit section of the canal. As is well known the rainfall in that drainage area is not ideally distributed throughout the year. The supply is ample in gross quantity, but is so distributed as to provide an excess of water during the rainy season and, generally, a shortage during the four months of the dry season. In other words, the result is that the maximum amount of traffic can always be accommodated during the rainy season—water is sufficient for all lockages—but it may be limited during the dry months. It has been estimated reliably that with even the dryest of dry seasons, the existing canal facilities can accommodate from 30 000 000 to 40 000 000 tons of shipping per year without interruption of service.*

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* "The Panama Canal After Ten Years", *The Military Engineer*, May–June, 1925.

† "The Panama Canal," by Messrs. Bakenhus, Knapp, and Johnson.

Contrary to current popular belief, this limitation is not a new discovery, nor an evidence of improper design and lack of forethought on the part of the builders. Neither is it a serious menace to the continued operation of the canal under any conditions of traffic which have been encountered to date. The characteristics of the rainfall in the Lake Gatun area were well known and fully considered when the canal was designed. It was realized that construction of an additional dam to create another storage reservoir in the Chagres River Valley above Lake Gatun would be necessary before the ultimate capacity of the canal as determined by the maximum number of lockages could be utilized. Accordingly, hydrological and other data have been compiled for years in anticipation of the inevitable dam and reservoir construction at Alhajuela, Panama, just outside (east of) the Canal Zone.

The expected traffic in the early years of operation did not require the additional storage basin for maintaining the lake level, so that this development was very wisely left for later years. Since annual traffic is now approaching the expected stage, and since no reasons exist for believing that the Panama seasons will change their rainfall characteristics, it is now advisable and justifiable to appropriate funds for the long considered Alhajuela Dam construction.† Other benefits, such as hydro-electric power, would also result from this project, which involves no particularly difficult engineering problems. Four or five years' time for construction and an expenditure of $10 000 000 to $15 000 000 will insure the continued operation of the canal under the maximum traffic load of 80 000 000 tons annually, regardless of the fluctuations of the rainfall.‡

Records of Panama Traffic.—Having analyzed the physical factors which limit the capacity of the Panama Canal, the next logical step is to review the history of canal traffic. Table 1, from the official records of the Panama Canal, shows the amount of shipping in net tons through the canal during each calendar year since its opening. These data do not include U. S. Naval vessels and miscellaneous craft exempt from tolls. It is obvious that, during the year of heaviest traffic to date (1927), the 26 600 000 tons taxed the canal capacity to about one-third of its maximum of 80 000 000 tons per year. This fact is entirely reconcilable with the operating policy of the canal, that is, to receive ships for transit during only 8 hours of the 24-hour day. In other words, the canal is regularly operating at full speed and to maximum capacity during the one-third of the time traffic is accepted.

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* "The Construction of the Panama Canal," by Sibert and Stevens.

† The 1929 Army Appropriation Bill, H. R. 10 286, approved March 23, 1928, contains the following item:

"$250 000 for commencing the construction of a dam across the Chagres River at Alhajuela for the storage of water for use in the maintenance and operation of the Panama Canal, together with a hydro-electric plant, roadways, and such other work as in the judgment of the Governor of Panama Canal may be necessary, to cost in the aggregate not to exceed $12 000 000."

‡ "The Panama Canal," by H. F. Hodges, Maj. Gen., U. S. A. (*Retired*), M. Am. Soc. C. E., *Professional Memoirs*, U. S. Corps of Engrs., October–December, 1909.

TABLE 1.—PANAMA CANAL TRAFFIC.

Calendar year.	Net tonnage.
1914*	1 284 293
1915†	3 902 592
1916†	3 817 704
1917	6 217 054
1918	6 409 886
1919	6 943 087
1920	10 378 265
1921	11 435 811
1922	12 992 573
1923	24 737 437
1924	24 411 760
1925	22 958 158
1926	25 836 241
1927	28 610 984

* Canal opened August 15, 1914.
† Canal closed three months by slides.

In addition, during some of the heaviest months of commercial traffic (and during the dry season as well) the bulk of the American fleet has been expeditiously passed through the canal without interfering with routine commercial traffic, simply by increasing the number of operating hours. Regularly the locks have been placed out of commission for overhaul and repair. Yet actually the minimum lake level has never been reached. The Panama Canal has operated and is operating regularly and comfortably at less than one-third the ultimate capacity of the present installation of locks. Use of less than one-third capacity, therefore, can hardly be termed a congested condition nor a situation which presages early inadequacy of the existing facilities.

Comparison of Suez Traffic.—The final step in foretelling the exhaustion of surplus capacity lies in predicting the future from the past and present. Dr. Emory R. Johnson, Dean of the Wharton School of Commerce and Finance of the University of Pennsylvania, who was a member of the original Isthmian Canal Commission, faced a similar problem in the preparation of his report on Panama Canal traffic and tolls as Special Commissioner for the President in 1912. Profiting from his experience and personal advice, two different but related criteria are introduced to study the probable future trend and growth of canal traffic. The first is the history of traffic through the Suez Canal, which is of much longer record than Panama and is singularly similar as an artificial highway of international shipping. The other basis is that of world commerce, or the combined exports and imports of the principal nations of the globe, which furnishes a fairly reliable barometer of the shipping through both Suez and Panama.

The annual Suez Canal traffic from the opening, and the annual total value of the world's commerce in billions of dollars for the same period are given in Table 2. The Suez statistics are taken from the official Suez Canal

Bulletin. The source of the world commerce figures is the Statistical Abstract compiled annually by the U. S. Department of Commerce.

The statistics of Tables 1 and 2 (Panama traffic, Suez traffic, and world commerce) are charted and presented graphically in Fig 1. The salient points should be noted—the apparent smoothness and gradual upward increase of Suez traffic (Curve *A*) from the opening of the canal in 1870 until the beginning of the World War in 1914; the world commerce curve (Curve *B*), similarly smooth and gradually increasing over the same period of time, although of course it does not originate at zero in 1870; the terrific jar to the world's equilibrium caused by the World War and reflected by the extreme peaks and valleys in the curve between 1914 and 1918; the rapid and steady climb in Panama Canal traffic during the canal's first decade of operation, as evidenced by the steep general slope of the Panama curve (Curve *C*); finally, and interestingly, too, the Panama traffic in 1926, practically equal to the Suez traffic. The vertical scale for charting world commerce having been suitably chosen, all three of the curves under consideration virtually attain a common point in 1926, which, in turn, permits of starting at a common origin in projecting the curves into the future.

TABLE 2.—SUEZ CANAL TRAFFIC AND WORLD COMMERCE.

Year.	Suez Canal, in net tons.	World commerce, in million dollars.	Year.	Suez Canal, in net tons.	World commerce, in million dollars.
1870	436 609	$10 633	1899	9 895 630	......
1871	761 467	......	1900	9 738 152	$20 105
1872	1 160 743	......	1901	10 823 840	......
1873	1 367 768	......	1902	11 248 413	......
1874	1 631 650	......	1903	11 907 288	......
1875	2 009 984	......	1904	13 401 835	......
1876	2 096 772	......	1905	13 134 105	......
1877	2 355 448	......	1906	13 445 504	27 418
1878	2 269 678	......	1907	14 728 434	......
1879	2 263 332	......	1908	13 633 283	......
1880	3 057 421	......	1909	15 407 527	......
1881	4 136 780	14 761	1910	16 581 898	33 364
1882	5 074 804	......	1911	18 324 794	35 909
1883	5 775 862	......	1912	20 275 120	39 750
1884	5 871 501	......	1913	20 033 884	40 420
1885	6 335 752	......	1914	19 409 495	37 760
1886	5 767 656	......	1915	15 266 155	31 302
1887	5 903 024	......	1916	12 325 347	46 523
1888	6 640 834	......	1917	8 368 918	52 781
1889	6 783 187	......	1918	9 251 601	62 802
1890	6 890 094	......	1919	16 013 802	75 811
1891	8 698 777	17 519	1920	17 574 657	61 277
1892	7 712 029	......	1921	18 118 999	61 417
1893	7 659 068	......	1922	20 743 245	46 137
1894	8 039 175	......	1923	22 730 162	49 918
1895	8 448 383	......	1924	25 109 882	50 782
1896	8 560 284	......	1925	26 761 935	57 608
1897	7 899 374	......	1926	26 060 377	55 000*
1898	9 238 603	......	1927	28 965 000	......

* Estimated ; complete official figures for 1926 not available.

Whereas Fig. 1 is an authentic graphic portrayal of official statistics, Fig. 2 is partly authentic, partly artificially manipulated, and partly conjectural. The Suez traffic (Curve *A*) and the world commerce curve (Curve *B*) are true copies from Curves *A* and *B* of Fig. 1 for the periods 1870 to 1913 and 1921 to 1926. The World War period, including the years of restoration to normalcy, 1914 to 1920, has been arbitrarily and artificially effaced from Fig. 2 by the simple mechanical expedient of plotting the year 1921 to follow immediately after the year 1913. For convenience a new curve, Curve *D*, has been introduced representing Suez Canal traffic averaged by decades. This smoothes out some of the minor irregularities in the Suez yearly curve. The last decade for Curve *D* also eliminates the World War period and averages the annual traffic for the ten years, 1910 to 1913 and 1921 to 1926, inclusive.

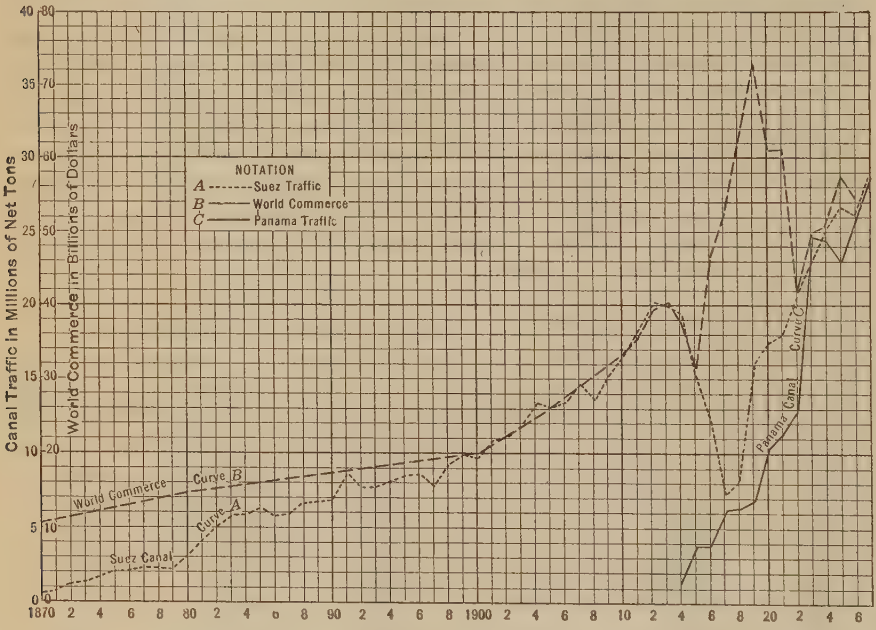

FIG. 1.

Future Trends.—Now comes the rub! The prolongation of a known curve into the unknown future is at once a dangerous and difficult undertaking, and, as has often been caustically said, "figures won't lie but liars will figure". The extension of a curve beyond its known values is particularly precarious and its reliability consequently is subject to wide differences of opinion when the length of the projection considerably exceeds the length of the established record. Predicting future behavior over a 5 or 10-year period from a given record of 50 years usually can be done fairly safely. A similar projection of a much shorter curve has many pitfalls; and a 50-year prolongation of a 15-year curve is like shooting at the moon. It can be done, however, and in some respects it is less complicated than making a similar projection for the much shorter 5 or 10-year period.

In attempting to forecast for the next decade a thorough analysis of political, economic, and social conditions throughout the world would be necessary to determine their probable trend and corresponding probable effect on canal traffic. Temporary local disturbances and commercial fluctuations would have to be carefully studied in a prediction covering only the next few years. However, in considering the next 50 to 100 years, unexpected, unknown, unpredictable factors may arise, such as world-wide disturbances, development of new trade centers and routes, revolutionizing inventions, etc., which events may entirely overshadow the effect of any expected, known, and predictable factors. In this prognosis of canal traffic over a relatively long time, therefore, no attempt is made to analyze or predict the conditions which will affect future Isthmian canal traffic. The forecast is made virtually entirely on the background of established history and tendencies.

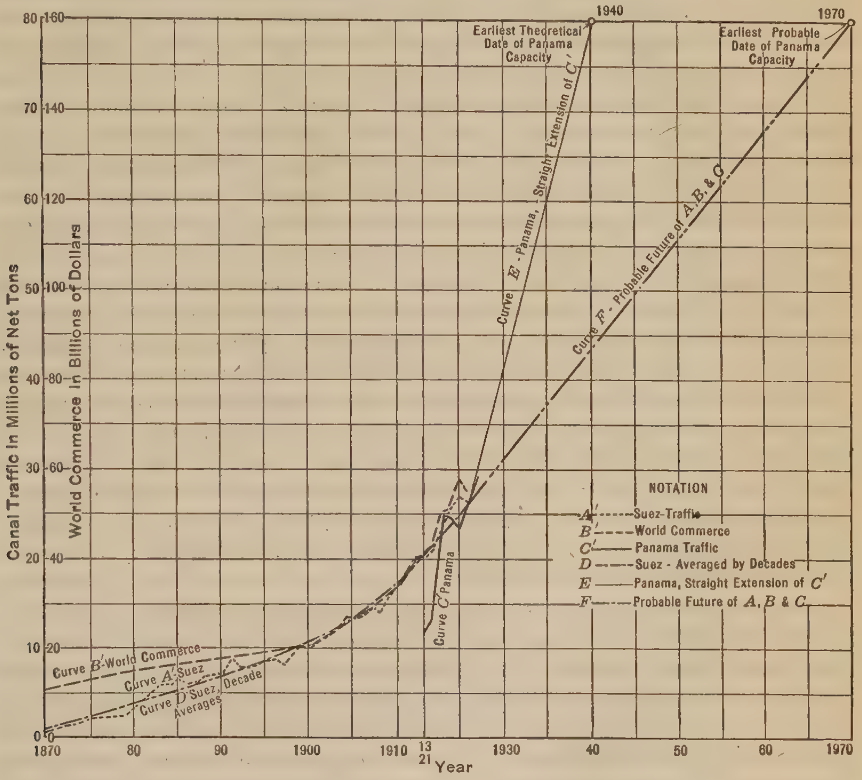

FIG. 2.

Straight-Line Projection of Traffic Record.—Referring to Fig. 1, it is evident that a purely theoretical prolongation of the Panama Canal Curve C would be practically a straight line of very steep slope. In Fig. 2 this theoretical extension has been plotted (Curve E). It intersects the upper horizontal line of the graph (the maximum ultimate capacity of the canal) at about 1940. That the traffic through the canal will continue to increase at its past rate so as to reach the saturation point in 1940 seems highly improbable. The creation of new devices usually results in rapid growth of utilization and in every healthy case such growth continues but generally at a more moderate rate after the novelty has worn off. Table 1, in fact, verifies this general law of behavior as applied to the Panama Canal, for the statistics of traffic since 1923 indicate normal and steady conditions after the rapid increase of the first decade. This theoretical straight-line extension (Curve *E*) of the Panama Canal traffic curve can be interpreted as the extreme condition in growth of traffic which would result in reaching the maximum canal capacity within the next fifteen years, if the rate of increase during the first decade continues—a highly improbable condition.

A condition more likely to prevail over a long period of future growth is that the present annual increment will remain constant, on the assumption that the rate has probably reached its maximum during the past few years. Such an assumed condition is portrayed graphically by a straight-line projection tangent to the curve at its last known point. As the 1926 point is virtually common to all three curves (Fig. 1), it affords a common origin in the speculative projection of the already artificially modified curves of Fig. 2. Curve *D,* showing Suez Canal traffic averaged by decades, is arbitrarily selected for the construction of the tangent straight line, Projection *F*. No high degree of mathematical exactness or mechanical precision has been resorted to in constructing this tangent, for such refinements would obviously be inconsistent with the degree of accuracy to be expected in the result. Line *F,* then, which represents simultaneously a theoretical prognosis* of Panama Canal traffic, Suez Canal traffic, and world commerce, intersects the saturation line of the Panama Canal (80 000 000 tons per year) at 1970.

Panama Canal Adequate Until 1970.—Let it be stated unequivocally, to forestall misinterpretation, that this is not a prediction that the Panama Canal will actually attain its ultimate maximum traffic in 1970. The problem of answering the "when" question is too difficult to permit of such a definitely accurate answer. The result obtained is, therefore, the probably propective earliest date at which the ultimate maximum capacity of the present canal facilities will be reached.

The 1970 intersection is the result of an assumed straight-line projection. That such a high rate of increase will continue uniformly for the next fifty years is, of course, possible, but it is also more than likely that, while traffic will continue to increase, its rate of growth will decline. In other words, the curve of the future is very apt to reach a point of inflection where it will depart from its straight upward course and tend to become more nearly horizontal. This tendency can reasonably be expected in the higher range of the diagram, justifying the inference that the straight-line tangent projection is reasonably conservative in predicting growth over a long period of time. Had the projection been actually made on the true curves of Fig. 1, rather than on the artificial curves of Fig. 2, it is evident that the slope of Curve *F* would have been somewhat nearer the horizontal and there would have been a resultant later intersection with the upper saturation line. This further emphasizes the conservatism of the 1970 intersection.

If the accuracy of established records, the reasonableness of assumptions, and the conservatism of construction methods, are acknowledged, then the

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* Traffic statistics for the calendar year 1927 are as follows:

Panama Canal..................	28 610 984 net tons
Suez Canal......................	28 965 000 net tons

result obtained should be accepted. It is believed that all these various factors have been satisfactorily explained and established. The conclusion therefore is warranted that the Panama Canal (augmented by the justifiable Alhajuela Reservoir development) will probably not be taxed to its maximum capacity within the next forty years, and that it will prove adequate to handle all probable traffic until the calendar reads 1970.

PROVISION FOR ADDITIONAL CAPACITY

What additional canal facilities should be provided when the present installation reaches its capacity? This second question is as important as the first and follows it most logically.

Having deduced that the present canal is adequate to handle all probable traffic for at least forty years, it seems somewhat paradoxical at this time to open any discussion of additional Isthmian canal facilities. Even granting that such a discussion is relevant, it is obvious that any comparative analysis of possible expansion projects must be made as of 1927. Many of the factors entering into such an analysis may remain materially unchanged over a long period of time, but others may be markedly altered as to character and effect during the next forty years. Physical conditions may remain practically as they are, but political conditions, particularly in Central America, are constantly changing their complexion. Social and economic factors, too, are not permanent, but are likely to vary in aspect and influence from time to time. In the ensuing discussion, therefore, the unavoidable inconsistency of presenting the 1970 situation from the 1927 viewpoint must be kept in mind.

Alternative Possibilities.—However, let it be assumed, for the sake of argument, that the Alhajuela Dam has been built and that the Panama Canal is carrying its maximum load of traffic. What additional facilities are now feasible? In all present-day discussions, fortunately only two alternatives are recognized. All Isthmian canal routes other than by way of Panama or Nicaragua were eliminated from consideration many years ago. Either supplementary sets of locks can be installed at Panama or a new canal can be constructed through Nicaragua.

The expansion at Panama would involve the construction of a third and perhaps, also, a fourth set of locks paralleling the existing twin-lock installation. The effect of such improvement would be the creation of a highway accommodating three or four lines of traffic where only two can flow at present. This enlargement would naturally entail some minor modifications in the adjoining existing structures, but would not involve any duplications or major changes in the other canal essentials, namely, Gatun Lake, the summit pool, Gatun Dam which impounds the lake water, Culebra Cut, the "big ditch" through the Continental Divide, and the terminal harbor facilities at Cristobal and Balboa. With the concurrent development of all feasible plans for increased storage and conservation of water no apprehension need exist as to any probable dry-season water shortage. The Nicaraguan project involves complete new construction starting on the Atlantic side in the vicinity of Greytown or San Juan del Norte, canalizing the San Juan River along the Nicaragua-Costa Rica border, through Lake Nicaragua, piercing the Continental Divide, and emerging at the Pacific side in the vicinity of Brito.

Pros and Cons of Panama and Nicaragua.—The advantages and disadvantages of the Panama and Nicaragua routes were exhaustively studied by the Isthmian Canal Commission and other eminent authorities before the Panama Canal was built. In the present comparative study of the merits of the two practicable expansion projects it is not intended to repeat long extracts from the Commission's authoritative report, nor to re-open the discussion of mooted questions which found their answers in the monumental accomplishments of the late George W. Goethals, M. Am. Soc. C. E., the late General Gorgas, and others. These records, written and physical, speak for themselves. For convenience the comparison of the feasible expansion projects at Panama and Nicaragua has been compiled in simple form in Table 3.

To accord with the classification of reasons in the discussion of the probable time of exhaustion of present facilities the comparative factors in Table 3 are grouped under technical and non-technical headings. The more important of the items will be elaborated. Although the factors listed undoubtedly do not exhaust the entire field, yet they are sufficiently complete for the present purposes. They are, in general, broad enough individually to embrace other incidental considerations and comprehensive enough collectively to cover the essential features.

TABLE 3.—COMPARISON OF EXPANSION PROJECTS.*

Item. (1)	Panama. (2)	Nicaragua. (3)
Technical:		
Type of canal	Lock (85-ft. lake level).	Lock (110-ft. lake level).
Length of canal	50 miles.	175 miles ±.
Terminal harbors	Excellent.	Pacific, fair; Atlantic, poor.
Navigation	Easy.	Difficult.
Length of steamship routes } offset.	Intercoast somewhat longer.	No advantage for South American trade.
Time of transit	6 to 8 hours.	24 to 30 hours.
Earthquakes	Not a serious factor.	More serious than Panama.
Construction facilities	Excellent existing.	None existing.
Additional capacity	40 000 000 to 75 000 000 tons (?).	80 000 000 tons (twin lock) (?).
Cost	$150 000 000 to $250 000 000 (?).	$500 000 000 to $1 000 000 000 (?).
Time to complete	8 to 10 years (?).	15 years (?).
Non-Technical:		
Defense—separate	Vulnerable.	Vulnerable.
Defense—simultaneous	Very difficult.	
Central American trade	No further effect.	Slight development.
Construction rights	Perpetual.	Perpetual.
Political situation	Stable.	Unstable.

* Corroborative evidence of reliability of the comparisons in Columns (2) and (3) can be found in the Official Report of the Isthmian Canal Commission, 1899–1901, and in "Problems of the Panama Canal," by Gen. Henry L. Abbot.

Comparison of Physical Features.—Frequent misstatements have been made regarding some of the inherent physical characteristics of the two routes. The Panama Canal, it is well known, is of the lock type with a nominal lake summit level of 85 ft. above the sea. The difference in elevation is overcome by twin flights of three locks at each end. Suitable foundations are known to be available for the construction of a third set of locks and may exist for even a fourth set. From deep water to deep water at both ends the Panama Canal has a total length of 50 miles, of which 24 miles are through Lake Gatun and 8 miles through Culebra Cut.

The question of lock *versus* sea-level design was widely discussed during Panama Canal construction days; but it applied only to the Panama route. Newspapers and magazines to-day refer nonchalantly to the advantages of a lockless sea-level canal by way of Nicaragua. Such references are little short of absurd. Whereas, the Panama route did originally offer a choice between the sea-level and lock alternatives, the Nicaragua route is universally recognized by the Engineering Profession to be impracticable for anything but the lock type of construction.

Proposed plans for a canal *via* Nicaragua contemplate the utilization of the large existing inland body of water, Lake Nicaragua. This would necessitate the maintenance of a nominal summit lake level of approximately 110 ft. above the sea. At least three steps of locks, as at Panama, and probably four, would be required to negotiate this difference in elevation. The total length of the Nicaragua route is about 185 miles, of which about 100 miles would be along the crooked San Juan River which would require artificial canalization. About 70 miles would be through Lake Nicaragua itself which would include about 20 miles of channel dredged (and continuously maintained) through the soft mud and fine silt in the shallow southwestern part of the lake. The severing of the remaining obstacle, the Continental Divide near the Pacific end, would require a cut perhaps 20 miles in length. This brief summary of the outstanding physical features of the two routes is sufficient to indicate that the Nicaragua project would be an even more stupendous undertaking than the construction of the Panama Canal. This prodigiousness, of course, would be reflected in the ultimate cost of construction.

In comparing the item of terminal harbors Panama possesses a wide advantage. The existing harbors at Cristobal and Balboa are excellent examples of the most modern type of port development. At Nicaragua, two harbors remain to be created. The one at the Pacific end could probably be built at a reasonable cost, but the site on the Atlantic presents unusual natural difficulties both for construction and maintenance.

Relative Advantages for Navigation.—The difficulties to be met in actual navigation are an important consideration. The gentle curvature of the channel of the Panama route permits of easy negotiation. The Nicaragua channel would be much more tortuous. The relative lengths of the two channels, has a weighty bearing on the question of navigation difficulties, particularly in view of the necessity for night traffic. Much more severe rainfall than is prevalent in Panama is to be expected in Nicaragua. Then, too, the heavier trade winds and stronger channel currents will be encountered. The navigator's choice would undoubtedly favor Panama.

The most ardent advocates of the Nicaragua route, both past and present, propose, as one of their strongest arguments, the advantage of the Nicaragua route in shortening travel distances (and hence time) between principal ports.

The validity of this argument is, however, more apparent than real. Where differences in distance favoring the Nicaragua route do exist, the apparent advantage is fully compensated by the difference in time required to traverse the canals and the inevitable high difference in insurance rates caused by the difference in navigation risks. As compared to Panama the actual sea distance by way of Nicaragua is 666 statute miles shorter between San Francisco, Calif., and New Orleans, La., and 434 miles shorter between San Francisco and New York, N. Y., a gain in time of approximately 48 hours and 31 hours, respectively, for the average modern steamship. Between the eastern coast of the United States and the western coast of South America—a much frequented trade route—the apparant advantage of shorter distance by way of Nicaragua disappears entirely for the eastern terminus of the Nicaragua route is actually farther from the Atlantic seaboard than the eastern end of the Panama Canal.

The time required for passing through the canals must, of course, be considered concurrently with the relative sea distances. At Panama from 6 to 8 hours are consumed in traversing the Isthmus. Estimates for Nicaragua vary, but the time may be conservatively placed at from 30 to 36 hours without allowing for probable frequent delays caused by forced retarding of operating speeds. The closer comparison of the routes, therefore, on the basis of economy of vessel operation, shows that the merits of the Nicaragua route in this respect are generally over-emphasized.

Feasibility of Constructions.—The relative possible effect of earthquakes on structures and excavations along the two routes is not considered of primary importance. At Panama the slides are no longer a problem and earth tremors have never been serious enough to affect other structures. What the history of the Nicaragua route would be in this respect is difficult to divine. The fallacy of a sea-level waterway having been previously shown, the claims for any additional inherent stability and permanency of this type of construction are entirely vitiated. Judging from past records and natural existing conditions along the route a lock canal at Nicaragua would be extremely fortunate to enjoy as uninterrupted an existence as the one at Panama.

The influence of the utter lack of construction facilities, such as a paralleling railroad along the Nicaragua route, is obviously included in the item of cost of the project. It should be noted too that the availability of plant and exact knowledge of working conditions at Panama permit of a cost estimate with a fairly high degree of accuracy, whereas the absence of all such facilities and the uncertainty of working conditions necessarily make the Nicaragua estimates much less reliable.

Cost Estimates Favor Panama.—Although specific estimates have been made as to the comparative costs in time and money of the two alternatives, the bases are mysteries. Since the report of the Isthmian Canal Commission in 1901 no further official investigation has been made nor estimate prepared of the Nicaragua project. Neither have any recent official estimates been prepared and published for additional lock construction at Panama. Estimates in the engineering sense generally imply preliminary analyses without which they are merely guesses. The currently quoted figures are undoubtedly merely offhand guesses which by repetition have been dignified by the appellation, "estimates".

The present lack of prepared estimates applies with equal force to both the Nicaragua and Panama expansion projects. The generally quoted figures are from \$150 000 000 to \$250 000 000 for additional lock construction at Panama and from \$500 000 000 to \$1 000 000 000 for a new canal *via* Nicaragua, with an estimated time of construction of 6 to 9 and 12 to 15 years, respectively. No mention is made of the additional tonnage capacity which will be provided by these expenditures. The amount of such increase will depend largely on the design adopted, but it may be presumed that both projects would provide about 100%, or 80 000 000 tons per year, additional capacity.

A simple attempt may be made to arrive at some estimates of cost slightly more reliable than mere guesses. The method is admittedly crude, but sufficiently satisfactory for the present rough comparative purposes. All details, some of which are of major importance, are purposely omitted from consideration. The report of the Isthmian Canal Commission in 1901, on which the decision between Panama and Nicaragua at that time was finally based, estimated the cost of the Panama project at \$144 000 000 and that of the Nicaragua project at \$190 000 000. As actually built the Panama Canal cost \$347 000 000. By simple proportion, taking into account the depreciation in the value of the dollar, the construction cost in Nicaragua in 1927 can be estimated at \$685 000 000. From the total cost of the Panama Canal it is difficult to segregate the cost of the locks as a separate item, but it approximated \$76 000 000. Again, by simple proportion, a 1927 duplication of the Panama locks can be estimated at \$125 000 000. Regardless of the absolute accuracy of these estimates they undoubtedly indicate the relative costs involved in choosing between the two alternatives.

A cursory summary of the advantages and disadvantages possessed by the two feasible expansion projects, as far as technical aspects are concerned, shows that the balance is decidedly in favor of the Panama alternative. From the unrelenting viewpoint of economics—dollars and cents—the Panama choice appears to be indisputable.

Non-Technical Arguments.—In the same way that factors other than technical have a bearing on the necessity for additional canal facilities, they must be given due consideration in comparing the feasible expansion projects. The order of their presentation is no indication of the relative importance of these items, for naturally their weight is neither constant nor commensurable.

Perhaps the most interesting of these items is the question of defense of the two routes. Being fundamentally of the same type of construction, it follows theoretically that in this respect the two alternatives will be equally vulnerable to hostile attack. The one presents perhaps a more concentrated target, the other a more extensive one. Both require fortification and defensive organization on a common scale.

The adage, "It is unwise to carry all your eggs in one basket", has been invoked to catch popular fancy as an argument for a Nicaragua Canal. The appropriateness of the proverb is questionable, however. The Isthmian canal situation is not a question of carrying eggs, but of protecting baskets. It has been found difficult to teach the American people the doctrine of reasonable military preparedness. War correspondents who have witnessed maneuvers in Panama have correctly pointed out the inadequacy of the canal defenses, particularly in their air aspect. The necessity for fortifying another site can hardly be expected to correct existing defects nor to simplify the problems of defense.

A word about commercial prospects: That some impetus to local business will result in the area contiguous to the Nicaragua Canal, if built, is certain. The field of development of Central American trade and industry may seem large, but is generally over-rated. Its possibilities certainly do not, in themselves, warrant any considerable expenditures. The Nicaragua route opens no new important markets or sources of raw material that are not equally served or reached by the Panama route.

International Factors.—The final element now deals with political conditions. The stability of the situation in Panama, as a virtual protectorate of the United States, is attested to by its relative tranquillity. The United States has exclusive perpetual canal rights in Panama. Although the United States also has exclusive perpetual rights for the construction of the Nicaragua Canal, obtained by treaty with Nicaragua in 1916 for a consideration of $3 000 000, the conditions for the Nicaragua route are not so favorable. The San Juan River which is the essential route for the canal between the Atlantic and Lake Nicaragua is along the international boundary between Costa Rica and Nicaragua. The establishment of a Canal Zone as a buffer State under United States jurisdiction would probably be welcomed or opposed by Central American strategists, depending on their motives. By some the construction of a canal *via* Nicaragua may be hailed as a panacea to cure all Central American ailments, but it is doubtful whether it would enjoy a very care-free existence in the troubled times of to-day.

CONCLUSION

The study of the predictable future growth of Isthmian canal traffic has indicated that the peak-load capacity of the Panama Canal will probably not be reached until 1970. The current discussions, in Congress and the press, therefore, appear to be a quarter of a century in advance of necessity. Not that foresight is ever to be condemned, but extensive preparations for a situation which may be materially altered by unforeseeable developments seems unjustifiable. Perhaps events to transpire in the future will place a different complexion on both the necessity and the advisability of a Nicaragua Canal Neither appear to exist at present. In short, the problem belongs to posterity.

DISCUSSION

William B. Landreth,* M. Am. Soc. C. E. (by letter).—The author has done a great service by giving a clear and concise statement of Isthmian Canal conditions. Taking the last four years in Table 1 as fairly typical of the yearly Panama tonnage, the annual percentage of gain or loss is found to be as follows: 1924, 1.30% less than 1923; 1925, 5.95% less than 1924; 1926, 12.60% more than 1925; and 1927, 10.70% more than 1926. On the basis of total tonnage, the four-year gain in 1927 is 15.65% of the 1923 tonnage.

The annual report of the State Superintendent of Public Works for 1927, gives the tonnage of the New York State Barge Canal for 1919 to 1927, inclusive. Making comparisons on the same basis as for Panama, the annual percentage of gain or loss is found to be as follows: 1924, 1.30% more than 1923; 1925, 15.30% more than 1924; 1926, 1.07% more than 1925; and 1927, 8.97% more than 1926. Likewise, on the basis of the total tonnage, the four-year gain in 1927 is found to be 28.69% of the 1923 tonnage.

During the canal season of 1928 the gain over that of 1927 has been 19.58 per cent. While the net tonnage on the Panama Canal is much greater than on the Barge Canal, the total gain on the Panama Canal for 1924 to 1927, inclusive, was 15.65% and on the Barge Canal for the same years it was 28.69 per cent. For two of the years the Panama Canal lost traffic, while the Barge Canal gained for each of the four years.

The Barge Canal passes through a populous section of the State and, therefore, a large number of pleasure boats pass through it. During the 1928 canal season, to October 13, the pleasure boats and their lockages were as shown in Table 4.

TABLE 4.—Statistics on the New York State Barge Canal for 1928.

Location.	Number of boats.	Number of lockages.
Near eastern end of Erie Canal	2 454	650
Near western end of Erie Canal	144	130
Near south end of Champlain Canal	1 528	674
North end of Champlain Canal	689	505
North end of Oswego Canal	251	167
Near north end of Cayuga-Seneca Canal	193	135
Total	5 259	2 261

Probably 15 000 to 20 000 people find enjoyment on the pleasure boats; the lockages used by them equals that used by several million tons of cargo. One should remember that the Panama Canal can be operated for 12 months each year while the weather prevents the use of the Barge Canal for more than about 60% of the year.

An interesting comparison is made between the Panama and Nicaragua Canals in Table 3, which includes a very concise statement of the salient points of the two routes. This table gives the time of transit at Panama as 6 to 8 hours and estimates it for the longer Nicaragua Canal at from 30 to 36 hours. This would mean a rate per mile of from 6.26 to 8.33 hours at Panama and from 4.86 to 5.83 hours at Nicaragua. The vertical rise at Nicaragua is about 30% more than at Panama, and differences in the navigation conditions would tend to make the rate at Nicaragua less than at Panama.

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On the Barge Canal the motor ship, *Twin Ports* (259 ft. long, 42 ft. wide, and loaded to a draft of 9 ft.), passed from Waterford to Oswego, N. Y.—a distance of 187 miles—in 39 hours and 22 min., which is an average speed of 4.75 miles per hour. The boat passed through 29 locks and 22 miles of lakes on the trip. Other similar boats have made the trip, going or coming, in 42 hours.

The estimate of from 30 to 36 hours for the transit through the 175 miles at Nicaragua seems to be conservative. The writer believes that the conclusions reached by the author are fair and that the paper will prove of great value, especially to those interested in canals.

EDUARDO ICAZA,* JUN. AM. SOC. C. E. (by letter).—The author has demonstrated beyond a shadow of a doubt that the logical method of supplying future needs for additional Isthmian Canal facilities (in so far as shipping is concerned) is the installation of more locks at Panama. Table 3 gives an excellent comparison. Under "Feasibility of Constructions", may be added the statement that sanitation in the Panama Canal Zone is more favorable than in any other area in the tropics.

One consideration has been omitted and that is the immediate and urgent need of a permanent non-movable steel highway bridge across the canal. It must not be forgotten that the canal has divided the Republic of Panama in two; that Panama has spent millions of dollars on new roads on both sides of the canal and that at present (1928) the only available means of crossing is (at one's own risk) by means of an inadequate ferry service.

Therefore, the Isthmian Canal situation is not wholly providing for future shipping. This, as the author has aptly stated, is a problem that "belongs to posterity". An important part of the present Isthmian Canal situation is to provide a non-movable bridge across the great waterway.

HANS KRAMER,† ASSOC. M. AM. SOC. C. E. (by letter).—An interesting comparison is drawn by Mr. Landreth which shows the relation between growth of traffic in recent years for the New York State Barge Canal and the Panama Canal. Similar comparisons might be drawn for the Sault Ste. Marie, Kiel, Manchester, and other large ship canals. The Panama and Suez Canals, however, are in a class apart in shortening trade routes by the perimeter of a continent. Hence, in the economic study of probable traffic growth, comparison has been limited to these canals, which alone represent essentially similar conditions.

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Mr. Icaza's suggestion that sanitary conditions in the Canal Zone are more favorable than those in Nicaragua, should be included in Table 3. His point regarding the need for a fixed highway bridge, although it is an important National problem for Panama, is not considered to be directly related to the adequacy of facilities for handling water-borne traffic.

An interesting review and a translation of a part of the writer's paper has been published* under the title, "Le Trafic Actuel et Futur du Canal de Panama". The review is prefaced by the editorial comment that the writer's conclusion is entirely optimistic. This criticism applies to the writer's prediction of the probable adequacy of the Panama Canal until 1970. In the ensuing French discussion, however, no definite errors of fact or assumption which lead to the 1970 conclusion are adduced to support the sweeping allegation. French engineers, because of de Lesseps' connection with the Panama Canal project, maintain a natural interest in the canal's present and future.

Since the preparation of the paper other reliable opinions have been recorded which tend to bear out the writer's conclusion that the rate of increase in traffic will be more moderate in the future. In testifying before a Sub-Committee of the House Committee on Appropriations on December 11, 1928, Harry Burgess, Colonel, Corps of Engineers, U. S. A., M. Am. Soc. C. E., Governor of the Canal Zone, stated that "as nearly as we can tell, the increase will not be more than ten million tons per decade". This forecast is in line with the "constant annual increment" assumption on which Curve *F*, Fig. 2, is based, and is even more conservative than the 12 500 000-ton increase per decade predicted by the curve.

Similar non-alarming conclusions are reached by the Standing Committee on "Maximum Capacity of Panama and Steps Being Taken by Those in Authority for Enlargement of the Canal for Shipping in the Future". This Committee reported† to the Association of Pacific and Far East Ports at its Fifteenth Annual Convention at Los Angeles, Calif., June 28, 1928: (1) That the canal was then being used to only one-third its capacity; (2) that the normal increase was less than 2% per year; (3) that, therefore, the canal would not be taxed to full capacity until about 1958 or later; (4) that the public should trust the Canal Administration to anticipate the need of enlargement or of building another canal; and (5) that the need for such expansion was assuredly far in the future.

The reasonableness of these viewpoints is supported by the actual Panama Canal record of transits for the calendar year, 1928. During this last year the net tonnage transmitted was 28 943 437, an increase of 332 453 tons, or 1.2% more than 1927 and an average increase of 1 553 598 tons per year, or 6.0% per year for the two-year period since 1926. The ordinate for the 1928 abscissa on Curve *F*, Fig. 2, predicts a probable traffic of 28 300 000 tons, which is within 2.1% of the actual record for the latest year.

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* *Le Génie Civil,* October 20, 1928.

† *World Ports,* September, 1928, p. 983.

After considerable debate in both Houses, the Seventieth Congress passed Senate Joint Resolution 117 which was approved March 2, 1929,

> "* * * authorizing an investigation and survey for the purpose of ascertaining the practicability and the approximate cost of constructing and maintaining additional locks and other facilities at the Panama Canal, and for the purpose of ascertaining the practicability and probable cost of constructing and maintaining an interoceanic ship canal across the Republic of Nicaragua".

The Chief of Engineers, under the direction of the Secretary of War, is formulating a policy for the execution of this mission. The forthcoming study should satisfy legislators, agitators, and the general public that there are no serious economic or engineering causes for alarm in the present Isthmian Canal situation.

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Paper No. 1734

CHARACTERISTICS OF CENTRIFUGAL OIL PUMP*

By Michael D. Aisenstein,† Esq.

With Discussion by Messrs. Morrough P. O'Brien and Michael D. Aisenstein.

Synopsis

The use of centrifugal pumps on oil pipe lines and at refineries has been growing. Because they are quantity pumps and have such merits as low first cost, simplicity of operation, steady flow, and adaptability for direct connection to high-speed machinery, centrifugal pumps attract all industries which have to deal with pumping machinery. Whereas the efficiency of a properly designed centrifugal water pump runs very high, this efficiency decreases when the same pump is used in a viscous fluid.

It is evident that, when pumping oil (besides all ordinary mechanical and hydraulic losses, for a viscous liquid), a new loss enters which dominates all the others. This is the viscosity or the internal resistance loss. The effect of viscosity is to decrease the head capacity and to increase the disk friction. This lowers the efficiency of the centrifugal pump correspondingly.

It is important to know the laws which govern the decrease in the head capacity and the increase of horse-power which affect the efficiency of the centrifugal pump. It is also important to know the limiting value of viscosity for each type and size of centrifugal pump at which it is practical to pump viscous oils. Moreover, as it is a comparatively easy matter to test a pump on water, it is of great advantage to have a method by which it is possible to predetermine what a centrifugal pump of known water charateristics will do when pumping a viscous oil.

By characteristics is meant the curves which show the relation between the capacity, head, efficiency, and power at a constant speed. These curves are usually obtained from an actual test using water as pumping medium.

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* Published in August, 1928, *Proceedings*.

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The original ideas which will be presented in this paper occurred to the writer while engaged in research work at the Hydraulic Laboratory of the University of California. These ideas, supplemented by numerous investigations made in the Test Laboratory of the Byron Jackson Pump Manufacturing Company, form the basis for this paper.

NOTATION

The following notation has been adopted:

d_2 = the diameter at outlet of impeller, in inches.
b_2 = the breadth between the impeller disks or shrouds at the diameter, d_2, in inches.
z = number of vanes.
A = the free exit area at the impeller periphery, in square inches = $(\pi d_2 - z t) b_2$.
D = the equivalent diameter of the free area at the impeller periphery $= \sqrt{\frac{4A}{\pi}}$, in inches.
ν_0 = kinematic viscosity of oil in poises.
ν_w = kinematic viscosity of water in poises = 0.01.
s = specific gravity.
t = vane length at periphery (outlet).
G_0 = U. S. gallons per minute when pumping oil.
G_w = U. S. gallons per minute when pumping water.
H_0 = head, in feet, when pumping oil.
H_w = head, in feet, when pumping water.
BHP_0 = brake-horse-power when pumping oil.
BHP_w = brake-horse-power when pumping water.
HP_f = horse-power absorbed to overcome the disk friction.
x = an exponent.
HP_m = horse-power absorbed to overcome journal and stuffing-box friction $= BHP_w(1 - e_m)$.
B = distance from the center of a disk to the wall of the casing.
e_m = mechanical efficiency.
T = torque, in foot-pounds.
e = pump efficiency.
U = peripheral velocity of the impeller, in feet per second.
g = gravitational constant = 32.2.
θ = angle.
ω = angular velocity.
n = an exponent.
r = radius.

GENERAL CONSIDERATIONS

The suitability of a pump for a given service is determined by the characteristic curves. A given pump operating at a constant speed can deliver a certain quantity of liquid at a given head. As the head is reduced the capacity increases. The maximum value of head occurs near the point of the zero discharge. Theoretically, the maximum head which can be produced by an impeller is,

$$H = \left[\frac{\text{Diameter, in inches} \times \text{revolutions per minute}}{1840}\right]^2$$

Actually, the maximum head may be more or less than the calculated value, depending mainly on the design of the impeller and of the casing. From given characteristics, new curves can be computed for speeds above and below the operating speed for the same pump. This is possible because if the diameter of the impeller is kept constant:

1.—The capacity varies as the first power of the speed;
2.—The head varies as the second power of the speed; and,
3.—The power varies as the third power of the speed.

It must be emphasized that this is correct if the variation in speed is not very great. Every pump is designed for a certain speed and it will not operate with the same efficiency at a speed very different from that for which it is designed.

Furthermore if the impeller speed is kept constant, provided the width of the impeller is constant:

(*a*) The capacity varies approximately as the diameter;
(*b*) The head varies as the square of the diameter; and
(*c*) The horse-power varies as the cube of the diameter.

These relations are true approximately. Actually, the head drops below the recalculated value. As a rule the efficiency of a pump falls off with the cut, although in some instances when the runner is too large for the casing the efficiency may improve for a relatively small cut.

These laws do not hold near points at which the pump cuts off; that is, points that mark sudden drops of head, efficiency, and horse-power. These drops occur when the suction inlet of the impeller is relatively small and the rate of discharge becomes too great for the net opening area of the impeller and when the pump becomes choked. No matter how wide open the valve on the discharge side of the pump is, the capacity stays constant.

Derivation of the Head-Capacity Equations

The effect of pumping different liquids with the same pump, operating at a constant speed, may be stated as follows:

1.—The capacity will vary inversely as some power of the kinematic viscosity of the liquid:

$$\frac{G_0}{G_w} = \frac{1}{\left(\frac{\nu_0}{\nu_w}\right)^n} \quad \text{(1)}$$

2.—The head will vary inversely as the square of the same power of kinematic viscosity of the liquid:

$$\frac{H_0}{H_w} = \frac{1}{\left(\frac{\nu_0}{\nu_w}\right)^{2n}} \quad \text{(2)}$$

Hence, the problem is to determine the various factors which affect the exponent, n.

The writer has found that n varies directly with the quantity, G, to some power, a, and the kinematic viscosity, ν, to some power, b. It varies inversely with the free exit area, A, to some power, c. Hence,

$$n = \frac{G_w^{\ a}\ \nu_0^{\ b}}{A^c} K \dots\dots (3)$$

Substituting the values:

$$A = (\pi\ d_2 - z\ t)\ b_2 = \frac{\pi\ D^2}{4} \dots\dots (4)$$

and,

$$K\left(\frac{4}{\pi}\right)^c = K' \dots\dots (5)$$

In Equation (3), the coefficient may then be written,

$$n = \frac{G_w^{\ a}\ \nu_0^{\ b}}{D^{2c}} K' \dots\dots (6)$$

This value of n may then be substituted in Equations (1) and (2), in order to obtain values of G_0 and H_0.

From experiments, the numerical values of the exponents were found to be:

$$K' = \frac{1}{15};\ b = 0.25;\ \text{and}\ 2\ c = 6\ \text{for}\ b_2 > \tfrac{1}{2}\ \text{in.}$$

For small values of b_2 the exponent, n, is very sensitive to any variation in the width of the impeller and it is always better to obtain this dimension by actual measurement. The value of $2\ c$ for $b_2 < \frac{1}{2}$ in. appeared to vary from 6 to 7. For viscosities of 1 600 S.s.u. (Saybolt seconds universal), and higher, $a = 1.5$. For viscosities less than this, a is determined from an empirical equation,

$$a = (\mu \times 0.1055)^{0.08} \dots\dots (7)$$

in which, μ is viscosity, in Saybolt seconds universal.

The net result is that Equation (6) becomes,

$$n = \frac{G_w^{\ a}\ \nu_0^{\ 0.25}}{15\ D^6} \dots\dots (8)$$

Analyzing Equations (1) and (2), it may be seen that $\dfrac{1}{\left(\dfrac{\nu_0}{\nu_w}\right)^n}$ is ordinarily less than unity, because $\dfrac{\nu_0}{\nu_w}$ for viscous oils is usually greater than 1.

In case $\dfrac{\nu_0}{\nu_w} = 1$, the term, $\left(\dfrac{\nu_0}{\nu_w}\right)^n$, becomes equal to unity, $G_w = G_0$, and $H_w = H_0$; that is, when liquids have the same kinematic viscosity, the head capacity curves are the same. On the other hand, the term, $\left(\dfrac{\nu_0}{\nu_w}\right)^n$, may become equal to unity when the exponent, n, is equal to zero. This is the case when $G_w = 0$; that is, when there is no flow, the "static head" at the shut-off valve is independent of viscosity and is approximately equal to $\frac{U^2}{2g}$. Since U is the peripheral speed of the impeller the "static head" depends on the speed of the prime mover and the dimensions of the impeller and is practically independent of the viscosity.

By studying the different factors that influence the exponent, n, it is possible to improve the design of a centrifugal oil pump. It may be easily noticed that the effect of viscosity on the head-capacity curve decreases rapidly with increases in the size of the pump.

DERIVATION OF THE HORSE-POWER EQUATIONS

Consider, next, the horse-power required by a centrifugal oil pump. In general, the horse-power taken by a centrifugal water pump is composed of (1) horse-power to overcome the disk friction; (2) hydraulic horse-power; and, (3) horse-power necessary to overcome the friction in the stuffing-boxes and bearings which, for a uniform speed, is practically constant.

The first two items increase with the viscosity of the liquid, and the brake horse-power of a pump operating in oil, may be expressed in terms of brake horse-power of a water pump by the following formula:

$$B\,H\,P_0 = s\,H\,P\,f_w \left(\frac{\nu_0}{\nu_w}\right)^X$$

$$+ (B\,H\,P_w - H\,P\,f_w - H\,P_m)\left(\frac{\nu_0}{\nu_w}\right)^n s + H\,P_m \ldots\ldots\ldots (9)$$

The factor, $H\,P\,f_w$, is the horse-power absorbed in overcoming the disk friction in water and is equal to the difference between $H\,P\,f_w$ corresponding to the outlet diameter of the impeller and the $H\,P\,f_w$ corresponding to the inlet diameter of the impeller.

Consider the first term of Equation (9); that is, the disk friction. When an impeller is revolving in a case of a centrifugal pump filled with any liquid, it acts as a brake. (See Fig. 1.) If U is the velocity of the element at the radius, r, and f is a coefficient of friction, the frictional resistance, R, equals:

$$R = f\,U^n A \ldots\ldots\ldots (10)$$

or the frictional resistance on an element of the area, $d\,A$,

$$d\,R = f\,U^n\,d\,A = f\,r^n\,\omega^n\,(r\,d\,\theta\,d\,r) \ldots\ldots\ldots (11)$$

The torque necessary to rotate the disk in liquid is,

$$d\,T = r\,d\,R = f\,r^{n+2}\,\omega^n\,d\,\theta\,d\,r \ldots\ldots\ldots (12)$$

$$T = \frac{4\,\pi\,f\,\omega^n}{n+3}\,r^{n+3}\text{ ft-lb} \ldots\ldots\ldots (13)$$

Taking into account the thickness, b,

$$T' = 2\,\pi\,r\,b\,f\,r\,r^n\,\omega^n = 2\,\pi\,b\,f\,r^{n+2}\,\omega^n \ldots\ldots\ldots (14)$$

$$T'' = T + T' = 2\,\pi\,f\,\omega^n\,r^{n+2}\left[\frac{2\,r}{n+3} + b\right]\text{ft-lb.}$$

and,

$$H\,P\,f = \frac{T''\,\omega}{550} = \frac{2\,\pi}{550}\,f\,\omega^{n+1}\,r^{n+2}\left[\frac{2\,r}{n+3} + b\right] \ldots\ldots\ldots (15)$$

The coefficient, f, is influenced by the ratio of $\frac{B}{2\,r}$, by the roughness of the surface, the viscosity, and the density of the liquid. Many empirical equations have been published which are a modification of Equation (15), namely, those of Unwin, Lashe, Banki, Gibson and Ryan, Le Conte, V. Zur Nedden, etc.

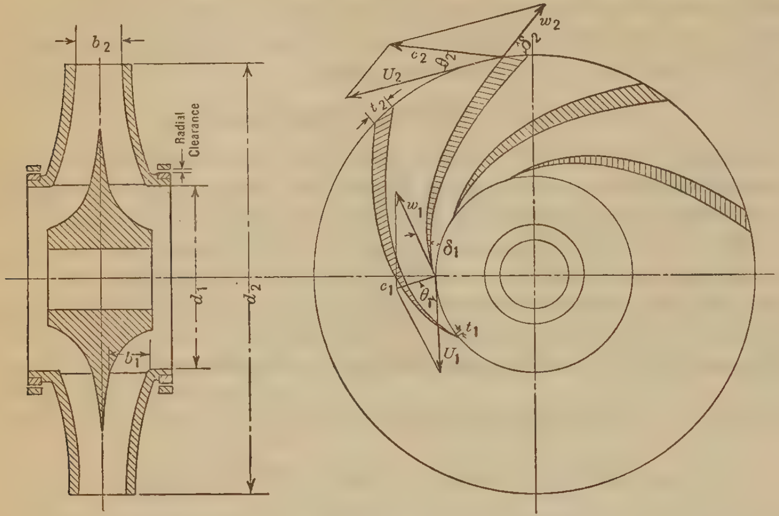
b_2
Radial Clearance
d_1
d_2
b_1
w_2
δ_2
c_2
β_2
U_2
t_2
w_1
δ_1
c_1
θ_1
t_1
U_1

FIG. 1.

Nearly all of them show that the horse-power absorbed to overcome the disk friction varies approximately as the fifth power of the diameter and as the third power of the speed. The writer uses Professor Banki's formula:

$$\text{Horse-power} = \frac{0.0000000961\ N^3\ d^5}{550}$$

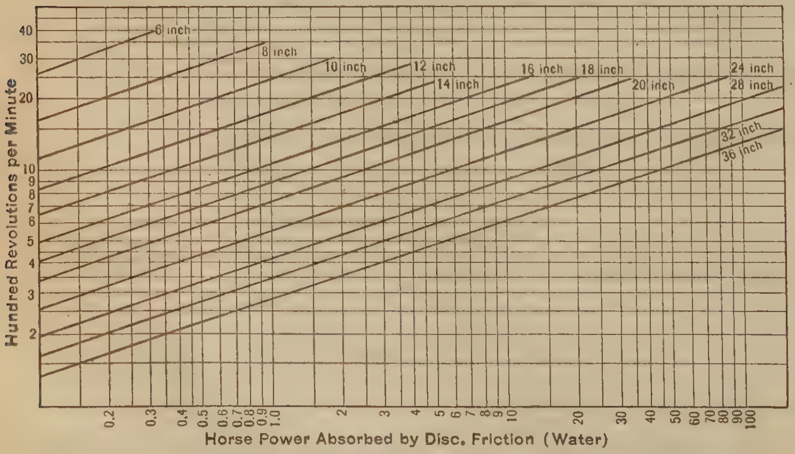

FIG. 2.—FROM PROFESSOR BANKI'S FORMULA FOR DISK FRICTION.

in which, N = revolutions per minute, and d = diameter, in feet. This is graphically represented in Fig. 2, which makes it possible to determine the $H\,P\,f_w$ direct for a given speed and diameter of the impeller in a pump with the usual clearances.

If an impeller is revolved at a constant speed in two different liquids, say, first in water and next in a viscous oil, then the relation between horse-power, kinematic viscosity, and specific gravity, is,

$$\frac{H\,P\,f_0}{H\,P\,f_w} = \left(\frac{\nu_0}{\nu_w}\right)^X \frac{s_0}{s_w} = \left(\frac{\nu_0}{\nu_w}\right)^X s_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The exponent, X, is a function of viscosity and is determined for a given viscosity from Fig. 3. This curve was plotted by the writer from experimental data. The second term of Equation (9) is the hydraulic horse-power. It is evident that, due to internal friction, it takes a greater amount of power to move a more viscous fluid and this can be expressed as:

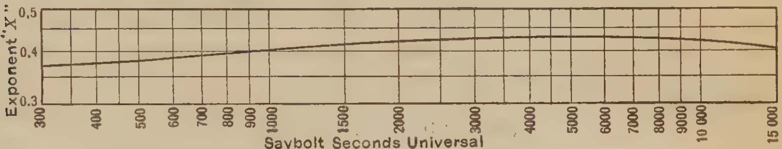

FIG. 3.—VALUES OF X TO BE USED IN EQUATION (9).

$$H\,H\,P_0 = (B\,H\,P_w - H\,P\,f_w - H\,P_m)\left(\frac{\nu_0}{\nu_w}\right)^n \quad \ldots\ldots\ldots\ldots (17)$$

in which, $(B\,H\,P_w - H\,P\,f_w - H\,P_m)$ is the hydraulic horse-power when pumping water. The exponent, n, was found to be the same as in Equation (6).

When designing a centrifugal pump it is always important to see that the percentage of horse-power absorbed in order to overcome the disk friction be a reasonable percentage of the total horse-power consumed by the pump. This is especially important when choosing an oil centrifugal pump, because the disk friction constitutes a tremendous loss in such a case, depending on the viscosity of the liquid. The problem of selection, the proper number of stages, dimensions, and angles of an impeller, must be studied very carefully.

In general, for a centrifugal water pump not more than 5% of the total horse-power is allowed for the disk friction. When the required head is high a single-stage pump would require either that the diameter of the runner be large or that the pump be run at a high speed. For this reason a multi-stage pump, with several runners of relatively small diameter, operating in series, is used.

In viscous liquids, the disk friction may be many times greater than in water, and it is always necessary to determine in advance whether a pump which is good for water service is suitable for oil and then select a proper motor.

Illustrative Example.—Consider a 5-in. single-stage pump operating at 1 750 rev. per min. Let $d_2 = 10\frac{1}{2}$ in.; $b_2 = \frac{3}{4}$ in.; $z = 8$; and, $t = 0.31$ in.

From Equation (4),

$$A = (\pi\, 10\tfrac{1}{2} - 8 \times 0.31)\, \tfrac{3}{4} = 22.9 \text{ sq. in.}$$

$$D^2 = 22.9\, \frac{4}{\pi} = 29.2$$

and,

$$15\, D^6 = 373\ 000$$

Next, predetermine the characteristics for the pump for oil with a viscosity of 675 S.s.u. and a specific gravity of 0.924. Then the kinematic viscosity in poises is computed by the empirical equation,

$$\nu_0 = \mu \times 0.00216 - \frac{1.8}{\mu} \quad \dots\dots\dots\dots\dots\dots (18)$$

$$\nu_0 = 675 \times 0.00216 - \frac{1.8}{675} = 1.457 \text{ poises}$$

$$\nu_0^{0.25} = 1.1$$

From Equation (7),

$$a = (675 \times 0.1055)^{0.08} = 1.405$$

The computations for the head capacity are shown in Table 1.

TABLE 1.

G_w.	H_w.	$G_w^{1.405}$.	n.	$\left(\frac{\nu_0}{\nu_w}\right)^n$.	$\left(\frac{\nu_0}{\nu_w}\right)^{2n}$.	G_0.	H_0.
880	45.0	18 509	0.0399	1.22	1.49	730	29.5
800	62.5	11 800	0.0349	1.19	1.42	678	44.0
600	85.0	7 760	0.0229	1.13	1.28	581	66.4
500	92.0	6 310	0.0186	1.10	1.21	454	76.0
400	97.0	4 570	0.0135	1.07	1.14	374	85.0
300	100.0	3 020	0.00891	1.06	1.12	283	89.3
200	100.8	1 699	0.00501	1.03	1.06	194	95.0
100	100.1	646	0.00191	1.01	1.02	99	98.0
0	99.0	0	0	1.00	1.00	0	99.0

Next, refigure the horse-power for the same viscosity and specific gravity. When a pump is lifting water, and is operating at full load,

$$B\,H\,P_w = 18.3$$

Assuming the mechanical efficiency of the pump to be 97%,

$$H\,P_m = B\,H\,P_w\,(1 - 0.97) = 0.55 \text{ h.p.}$$

Next, from Fig. 2, the horse-power absorbed to overcome the disk friction in water for an impeller, 10½ in. in diameter, revolving at 1 750 rev. per min., is:

$$H\,P\,f_w = 0.5$$

The $H\,P\,f_w$ for the inlet diameter of the impeller is negligible ($d_1 = 5$ in.).

From Fig. 3 the exponent, X, corresponding to a viscosity of 675 S.s.u., is:

$$X = 0.39$$

$$H\,P\,f_0 = 0.924 \times 0.5 \times 145.7^{0.39} = 3.23$$

TABLE 2.

G_0.	$B\,H\,P_w$.	$G_0^{1.405}$.	n.	$\left(\frac{\nu_0}{\nu_w}\right)^n$.	$s\,(B\,H\,P_w - H\,Pf_w - H\,P_m)\left(\frac{\nu_0}{\nu_w}\right)^n$.	$B\,H\,P_0$.
730	20.0	10 500	0.031	1.17	20.5	24.3
673	19.5	9 500	0.028	1.15	19.3	23.1
531	17.5	67 500	0.020	1.105	16.2	20.0
454	16.3	5 400	0.016	1.083	15.0	18.8
374	14.7	4 100	0.0121	1.062	13.2	17.0
283	13.0	2 800	0.0083	1.042	11.5	15.3
194	11.0	1 130	0.0033	1.017	9.5	13.3
99	9.0	630	0.0019	.....	7.7	11.5

Then tabulate the computations for $B\,H\,P_0$ in the same manner as for the head capacity. (See Table 2.)

Plotting the given values of H_0, $B\,H\,P_0$, and efficiency, against G_0 as abscissa, the oil-pumping characteristics of a centrifugal pump are found as based on water performance. Similar curves were plotted for viscosities of 1 350 and 3 400 S.s.u. (See Figs. 4, 5, and 6.)

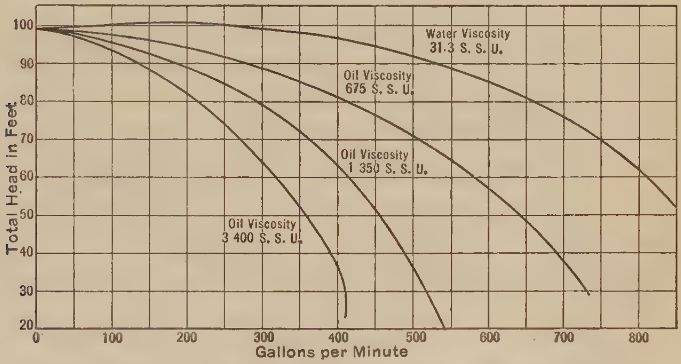

FIG. 4.—HEAD CAPACITY CURVES FOR A 5-INCH SINGLE-STAGE PUMP.

PROCEDURE IN COMPUTING THE OIL CHARACTERISTICS OF A MULTI-STAGE PUMP

The case of a multi-stage pump is more complicated than that of a single-stage pump. Oil passing through the first stage, on account of losses, is heated; and the more stages it has to pass, the more pronounced is the heating effect. The heat given to the oil raises its temperature. The change in temperature changes the viscosity, and the drop in head in the first stage will be greater than in the second stage; in the second stage greater than in the third, etc. Therefore, it is necessary, in the case of a multi-stage pump, to calculate the losses in the first stage and express them in British thermal units; determine the increase in temperature; look up the new viscosity corresponding to this temperature; and compute the head-capacity curve for the second stage, basing the calculations on the new viscosity which evidently is less than that for the first stage. After the head-capacity and brake-horse-power curves are obtained for each stage, it is necessary to add them to obtain characteristics of the entire pump.

This is a very long and tedious task and for ordinary practice it is sufficiently accurate to calculate the head capacity, brake horse-power, and efficiency for a given viscosity for one point, usually at the operating point, in the same manner as has been done for a single-stage pump. This will be the first approximation. The next step is to take the difference between the horse-power input in the pump and the horse-power output of the pump, convert this difference into British thermal units, and knowing the specific heat of the oil (which varies from 0.45 to 0.505), determine the rise in temperature and corresponding viscosity. With the new viscosity determined, the head-capacity, brake horse-power, and efficiency curves for the entire range may be calculated.

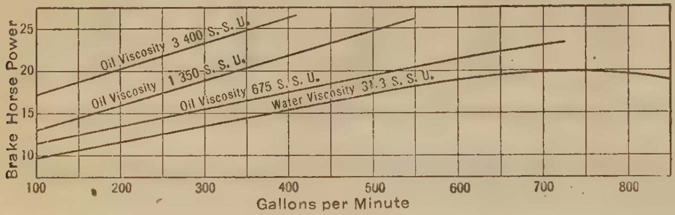

FIG. 5.—BRAKE-HORSE-POWER CURVES FOR A 5-INCH SINGLE-STAGE PUMP.

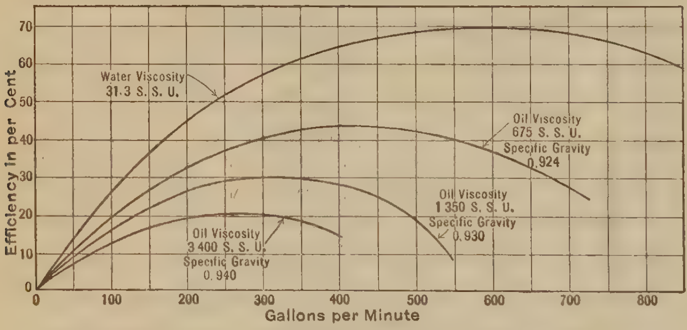

FIG. 6.—EFFICIENCY CURVES FOR A 5-INCH SINGLE-STAGE PUMP.

Illustrative Example.—Consider a four-stage pump operating at 1 750 rev. per min., which has the following dimensions: $d_2 = 15$ in.; $b_2 = \frac{1}{2}$ in., $t = 0$ in. (vanes filed to a point); and $z = 6$.

Then, from Equation (4),

$$A = \pi \times 15 \times \tfrac{1}{2} = 23.5$$

$$D^2 = 30$$

and,

$$15\ D^6 = 405\ 000$$

Let the temperature of the oil to be pumped be 104° Fahr. The viscosity corresponding to this temperature equals 1 000 S.s.u. (See Fig. 7.) Then,

$$\nu_0 = 1\,000 \times 0.00216 - \frac{1.8}{1\,000} = 2.16$$

$$\nu_0^{0.25} = 1.21$$

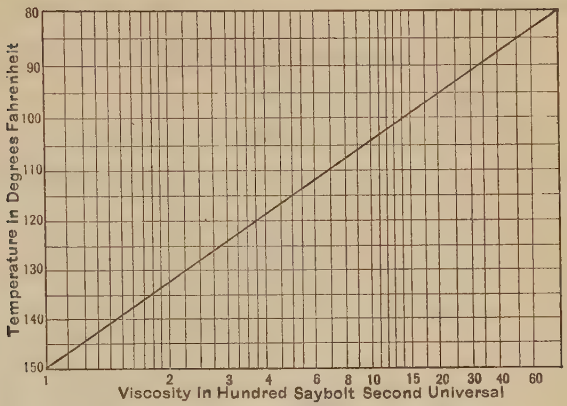

FIG. 7.—VISCOSITY-TEMPERATURE CURVE.

From Equation (7),

$$a = (1\,000 \times 0.1055)^{0.08} = 1.452$$

For the first approximation: $G_w = 600$; $H_w = 720$; $G_w^{1.452} = 10\,800$; $n = 0.0387$; $\left(\frac{\nu_0}{\nu_w}\right)^n = 1.23$; $\left(\frac{\nu_0}{\nu_w}\right)^{2n} = 1.51$; $H_0 = 476$; $G_0 = 487$; $G_0^{1.452} = 8\,000$; $n = 0.0239$; $\left(\frac{\nu_0}{\nu_w}\right)^n = 1.137$; and $BHP_w = 162$.

Assuming 97% mechanical efficiency of the pump and 0.95 specific gravity of the oil,

$$HP_m = 162\,(1 - 0.97) = 4.87 = \text{a constant}$$

HPf_w for an impeller 15 in. in diameter, revolving at 1 750 rev. per min., is 2.75 (from Fig. 2). Therefore, for four stages (neglecting HPf_w for the inlet diameter),

$$HPf_w = 4 \times 2.75 = 11$$

and the disk friction horse-power, when pumping oil, is found by Equation (16),

$$HPf_0 = 0.95 \times 11 \times 216^{0.402} = 90.2$$

and,

$$BHP_0 = 4.87 + 90.2 + (162 - 11 - 4.87)\,0.95 \times 1.137 = 252.7$$

The horse-power lost in heat is equal to,

$$BHP_0 - \frac{G_0 H_0 s}{3\,960} = 252.7 - \frac{487 \times 476 \times 0.95}{3\,960} = 197.2$$

The rise in temperature, in degrees Fahrenheit, is,

$$t_2 - t_1 = \frac{\text{Horse-power lost in heat} \times 42.44}{\text{Specific heat} \times G_0 \times \text{weight}}$$

In this case,

$$t_2 - t_1 = \frac{197.2 \times 42.44}{0.5 \times 487 \times 8.33 \times 0.95} = 4.3° \text{ Fahr.}$$

Hence, the new temperature is 108.3° Fahr., and the corresponding viscosity (from Fig. 7) is 800 S.s.u. Using this viscosity as the basis for the second approximation,

$$\nu_0 = 800 \times 0.00216 - \frac{1.8}{800} = 1.723$$

$$\nu_0^{0.25} = 1.145$$

$$a = (800 \times 0.1055)^{0.08} = 1.425$$

The computations for the head capacity are shown in Table 3.

TABLE 3.

G_w.	H_w.	$G_w{}^{1.425}$.	n.	$\left(\frac{\nu_0}{\nu_w}\right)^n$.	$\left(\frac{\nu_0}{\nu_w}\right)^{2n}$.	G_0.	H_0.
0	830	0	0	1.000	1.000	0	830
200	900	1 900	0.00536	1.028	1.057	195	850
400	845	5 100	0.0144	1.077	1.160	372	728
600	720	9 000	0.0254	1.140	1.300	527	552
800	515	10 350	0.0292	1.160	1.350	690	382

From Fig. 3, the exponent, X, corresponding to viscosity of 800 S.s.u. is 0.390, therefore,

$$H\,P\,f_0 = s \times H\,P f_w \left(\frac{\nu_0}{\nu_w}\right)^X = 0.95 \times 11 \times 172.3^{0.390} = 77.8$$

The computations for $B\,H\,P_0$ are given in Table 4. All values from Tables 3 and 4 are plotted in Fig. 8, and the characteristics of the pump for oil are obtained.

TABLE 4.

G_0.	$G_0{}^{1.425}$.	n.	$\left(\frac{\nu_0}{\nu_w}\right)^n$.	$B H P_w$.	$H P f_w$.	$H P f_0$.	$H P_m$.	$s\,(B\,H\,P_w - H P f_w - H\,P_m)\left(\frac{\nu_0}{\nu_w}\right)^n$.	$B\,H\,P_0$.
0	0	0	1.000	...	11	77.8	4.87	.....	.....
195	1 850	0.0052	1.027	110	11	77.8	4.87	97.5	180.2
372	4 500	0.0127	1.670	140	11	77.8	4.87	125.8	208.5
527	7 500	0.0212	1.105	170	11	77.8	4.87	161.9	244.6
690	11 000	0.0810	1.175	190	11	77.8	4.87	194.0	276.7

It must be mentioned that the point of cut-off and the lower part of the head-capacity curve, when calculated by the derived equations, may give values which are less than the actual. The reason for this is that the efficiency of the pump decreases toward the cut-off point and at this point practically the whole power is consumed in overcoming losses.

This means that there is a greater heating effect and, consequently, a greater decrease in viscosity at points closer to the cut-off point, and it is evident that the actual head-capacity curve may deviate from the theoretical head capacity which is based on constant viscosity.

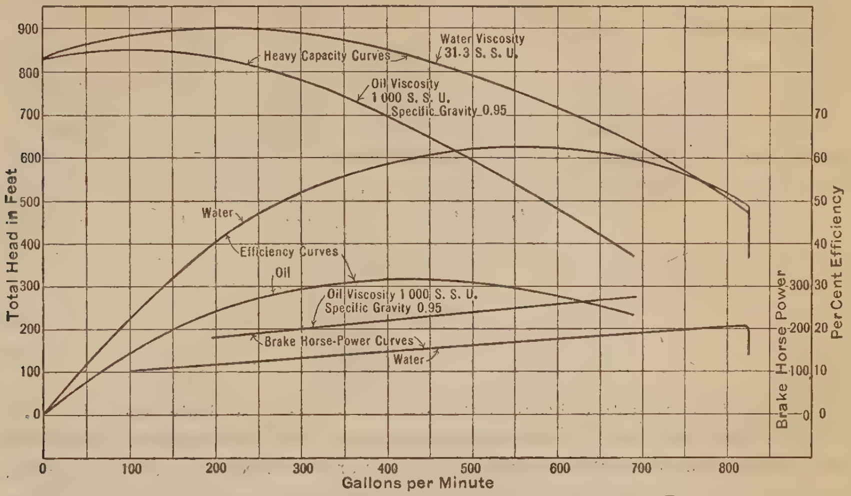

FIG. 8.—CHARACTERISTICS OF 3-INCH, 4-STAGE MULTIPLE PUMP.

Another point which must be emphasized is the effect of leakage. It is well known that the leakage past the runner wear-rings and the casing wear-rings of a centrifugal pump when lifting water is a relatively great loss, particularly for high heads and small pumps. It is evident that the leakage decreases with the viscosity, resulting in an increase of the volumetric efficiency of the pump. This probably is the reason for the increase of the exponent, c, for pumps of small sizes.

CALCULATION OF THE OIL CHARACTERISTICS FOR DIFFERENT SPEEDS

Suppose a pump had been tested at two speeds with oil and the performance from oil to oil was refigured for each speed. How do these oil performances for two different speeds compare? In other words, is it correct to refigure from a given performance of a pump on oil at one speed to another speed in the same way as is done for water?

Since in case of water the head varies as the square of the speed and the capacity as the first power of speed,

$$H_w = C_1 N^2 \qquad (19)$$

$$G_w = C_2 N \qquad (20)$$

and,

$$H_w = C G_w^2 \qquad (21)$$

In Equations (19), (20), and (21), N is the speed, in revolutions per minute, and C is a constant, for a given point, and is determinable for a given pump from the head-capacity curve at a given speed. Hence, if the speed is doubled, the capacity is also doubled, and the head is increased four times. In other words, when the pump is operated in water the reduction of a point at different speeds follows a line of relatively constant efficiency. This is a straight line with a slope of 2:1 when plotted on logarithmic paper.

This is not the case with oil, however. Expressing G_w (Equation (8)) in terms of speed,

$$n = \frac{(C_2\, N)^a\, \nu_w^{0.25}}{15\, D^6} \qquad (22)$$

The values expressed in Equations (19), (20), (21), and (22), may be substituted in Equations (1) and (2) as follows:

$$G_0 = C_2\, N \frac{1}{\left(\frac{\nu_0}{\nu_w}\right)^n} \qquad (23)$$

$$H_0 = C_1\, N^2 \frac{1}{\left(\frac{\nu_0}{\nu_w}\right)^n} \qquad (24)$$

In Equations (23) and (24), use the value of n given in Equation (22).

Since the exponent, n, is a function of speed, the head-capacity characteristics cannot be calculated in the same manner as for water. It is, therefore, necessary to refigure the speeds for water and then recalculate the performance from water to oil as shown. The same result will be obtained if the constants, C_1 and C_2, are determined from Equations (19) and (20) and the values of these constants and the speed, N, are substituted in Equations (23) and (24).

Illustrative Example.—Consider the 5-in. double-suction, single-stage pump which was used in Example 1 and calculate the characteristics for 1 450 rev. per min.

From Table 1,

$$G_w = 800;\ H_w = 62.5 \text{ at } 1\,750 \text{ rev. per min.}$$

From Equations (19) and (20),

$$C_2 = \frac{800}{1\,750} = 0.457$$

$$C_1 = \frac{62.5}{1\,750^2} = 0.0000205$$

By Equation (22),

$$n = \frac{(0.457 \times 1\,450)^{1.405} \times 1.1}{373\,000} = 0.02714$$

Substituting these values in Equations (23) and (24) and assuming that $N = 1\,450$ rev. per min.,

$$G_0 = 0.457 \times 1\,450 \frac{1}{(145.7)^{0.02714}}$$

$$G_0 = 577$$

$$H_0 = 0.0000205 \times 1\,450^2 \, \frac{1}{(145.7)^{2 \times 0.02714}}$$

$$H_0 = 32.7$$

In like fashion all the points can be determined and a smooth head-capacity curve can be passed through these points.

As an illustration of the effect of the viscosity on pumps of larger size, the performance of a 10-in. double-suction pump is plotted in Fig. 9 for water and for oil with a viscosity of 1 200 S.s.u.

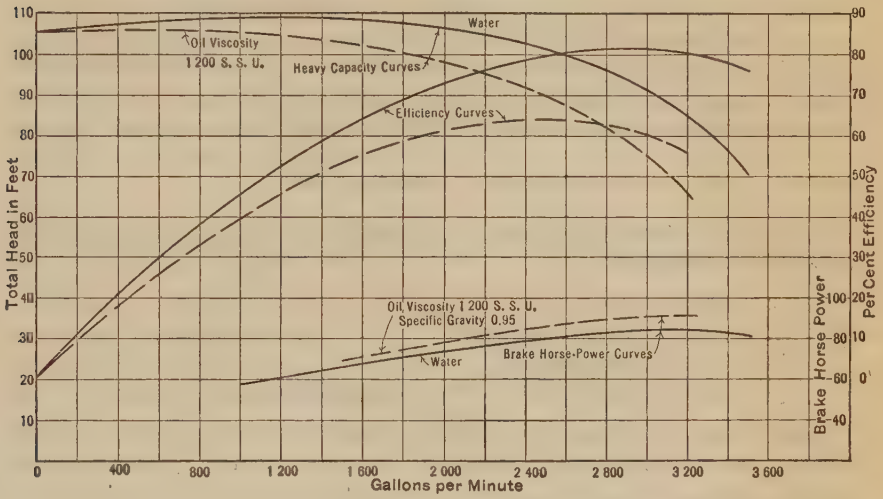

FIG. 9.—CHARACTERISTICS OF 10-INCH DOUBLE-SUCTION PUMP.

CONCLUSIONS

From these considerations, the following conclusions may be made:

1.—It is possible to predetermine, with a sufficient degree of accuracy, the performance of a centrifugal oil pump by observing its characteristics as obtained on water.

2.—The effect of the viscosity is to decrease the head capacity and increase the power.

3.—When the pump is working in viscous oil, the increase in horse-power is influenced mainly by the increase in the disk friction due to viscosity.

4.—The head-capacity characteristic and the efficiency of a centrifugal pump as applied to viscous oils improve with the increase of the size of the pump.

5.—The volumetric efficiency, particularly that of a low specific speed centrifugal pump, improves with the increase of the viscosity.

6.—In a multi-stage pump the performance is improved because of the heating of the oil in the pump by losses. This is more noticeable in pumps of low specific speed.

7.—It seems that an impeller with "flat" characteristics will be more efficient when pumping viscous oils than an impeller with "steep" characteristics.

DISCUSSION

MORROUGH P. O'BRIEN,* ASSOC. M. AM. SOC. C. E. (by letter).—The author's method of predicting the characteristics of an oil pump from its behavior when pumping water is somewhat similar to the method used in predicting the total resistance of a ship from tests of a geometrically similar model. The skin friction and the residual wave and eddy resistance are separated and stepped up to the full scale in different ratios, a procedure that is made necessary by the fact that the two resistances follow different laws of similarity. In the case of centrifugal pumps, the fluid is moving through closed conduits and hence no part of its resistance depends on the acceleration of gravity. For this reason, it should be possible to base the prediction of the performance with fluids of different viscosities on the Reynolds law of similarity which requires that the quantity, $\frac{v\,d}{\nu}$, be constant for dynamic similarity of flow.†

In this discussion, the notation is that used by the author, with the following necessary additions:

ρ = density of a liquid.
γ = weight per unit volume.
P = total power required.
p = power required to overcome bearing friction.

Considering two geometrically similar pumps of diameter, d and d_0, which are to be operated with fluids of kinematic viscosities, ν and ν_0, the condition for similarity is:

$$\frac{U\,d}{\nu} = \frac{U_0\,d_0}{\nu_0} \quad \text{.........................(25)}$$

in which, U and U_0 are the velocities at the points to which d is measured. Substituting for the velocities in terms of the quantity flowing and the areas,

$$\frac{G\,d}{A\nu} = \frac{G_0\,d_0}{A_0\,\nu_0}$$

Since the two pumps are geometrically similar, the areas are proportional to the squares of the diameters and,

$$\frac{G}{G_0} = \frac{d\,\nu}{d_0\,\nu_0} \quad \text{.........................(26)}$$

To obtain the relation between the speeds of rotation, the velocity, U, is replaced in Equation (25) by the angular velocity, ω, and the diameter, d, with the result that:

$$\omega = \omega_0 \left(\frac{d_0}{d}\right)^2 \frac{\nu}{\nu_0} \quad \text{.........................(27)}$$

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† "Die Grundlagen der Aehnlichkeits Mechanik and ihre Verwertung," Weber, Jahrbuch der Schiffbautechnischen Gesellschaft, 1919.

To determine the corresponding heads, the various components of the total head must be considered. For the pressure head, or head due to centrifugal action,

$$\frac{H'}{H_0'} = \frac{\rho\,\omega^2\,d^2\,\gamma_0}{\rho_0\,\omega_0^2\,d_0^2\,\gamma}$$

in which, H' is the pressure head of the liquid, in feet; ρ is the density of the liquid; and γ is the weight per unit volume. Considering geometrically similar volumes, V and V_0, similarly placed in the two pumps, the kinetic energy is:

$$K\,E = V\,\rho\,\frac{\nu^2}{2}, \text{ and } K\,E_0 = V_0\,\rho_0\,\frac{\nu_0^2}{2} \quad \text{(28)}$$

the velocity heads, or kinetic energy per unit weight, are,

$$H'' = \frac{V\,\rho\,\nu^2}{2\,V\,\gamma}, \text{ and } H_0'' = \frac{V_0\,\rho_0\,\nu_0^2}{2\,V_0\,\gamma_0} \quad \text{(29)}$$

and the ratio of velocity heads is,

$$\frac{H''}{H_0''} = \frac{\rho\,\nu^2\,\gamma_0}{\rho_0\,\nu_0^2\,\gamma} = \frac{\rho\,\omega^2\,d^2\,\gamma_0}{\rho_0\,\omega_0^2\,d_0^2\,\gamma} \quad \text{(30)}$$

Losses due to impact are proportional to the velocity head, and will be in the same ratio.

The remaining element of the total head to be considered is the loss due to friction. Experiments on the flow of fluids in pipe lines have shown that for equal values of the Reynolds number, the values of the friction factor are identical. This may be expressed as:

$$\frac{R}{\rho\,\nu^2} = \frac{R_0}{\rho_0\,\nu_0^2} \quad \text{(31)}$$

in which, R is the force per unit area resisting the motion. The resistance per unit area is proportional to the product of the loss of head, due to friction, H''', and the unit weight. Substituting this relation:

$$\frac{H'''\,\gamma}{H_0'''\,\gamma} = \frac{\rho\,\nu^2}{\rho_0\,\nu_0^2} \quad \text{(32)}$$

or,

$$\frac{H'''}{H_0'''} = \frac{\rho\,\omega^2\,d^2\,\gamma_0}{\rho_0\,\omega_0^2\,d_0^2\,\gamma} \quad \text{(33)}$$

It appears that all the elements of the total head are in the same ratio for dynamically similar conditions.

To test this method, the rotational speeds of the two geometrically similar pumps should be in the relation given by Equation (27). The heads should then be adjusted, until the quantities are in the ratio given by Equation (26). If the Reynolds law of similarity is true, the observed heads should then be in the ratio,

$$\frac{H}{H_0} = \frac{\rho\,\omega^2\,d^2\,\gamma_0}{\rho_0\,\omega_0^2\,d_0^2\,\gamma} \quad \text{(34)}$$

The relation between the total power, P, required is,

$$\frac{P - p}{P_0 - p} = \frac{\nu^3\,\rho\,d_0}{\nu_0^3\,\rho_0\,d} \quad \text{(35)}$$

in which, p is the power required to overcome bearing friction.

It should be noted that the geometric similarity of the two pumps should extend also to the surface roughness, which is a difficult condition to fulfill. For the purpose of checking this law of similarity, the impellers and casings might be given an artificial roughness proportional to the diameters. If the scale ratio is small, a rough agreement should be obtained by making both pumps of the same material.

If the scale ratio is unity, corresponding to the use of the same pump for both liquids, the speed ratio becomes,

$$\frac{\omega}{\omega_0} = \frac{\nu}{\nu_0} \qquad (36)$$

Should it be desirable to use the same speed of rotation, the diameters must be in the ratio,

$$\frac{d}{d_0} = \sqrt{\frac{\nu}{\nu_0}} \qquad (37)$$

The ratio of the kinematic viscosity of some common oils to that of water is about 18, making the fulfillment of Equation (36) difficult unless special testing equipment is used.

The author's method does not appear to be based on the condition of dynamic similarity of flow and, for this reason, it is applicable only in the range of types, diameters, and viscosities for which the coefficients have been determined. However, the method possesses the very definite advantage of simple experimental procedure. Unfortunately, the paper does not contain a comparison of the oil-pumping characteristics as predicted from water characteristics, and the actual oil-pumping characteristics as obtained by experiment.

Michael D. Aisenstein,* Esq. (by letter).—The writer agrees with Professor O'Brien's desire to use the Reynolds' law of similarity, which requires that the quantity, $\frac{v\,d}{\nu}$, be constant for dynamic similarity of flow; but several factors that make the application of Reynolds' criterion rather difficult should not be overlooked.

Argument No. 1.—Consider the various components of the total head:

$$H = h_w + h_{fw} + h_{sw} \qquad (38)$$

in which,

H = total theoretical head for finite number of vanes.
h = total head.
h_f = friction head.
h_s = loss in head due to sudden enlargement.

The subscripts, w, and o, denote water and oil, respectively.

If the flow throughout the pump is turbulent, the relative nature of the liquids pumped is immaterial theoretically and:

$$h_w + h_{fw} + h_{sw} = h_0 + h_{f0} + h_{s0} \qquad (39)$$

---

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Assuming for simplicity that the "shock loss" is not affected by viscosity,

$$h_{sw} = h_{s0}$$

and,

$$h_0 = h_w + h_{fw} - h_{f0} \quad \ldots\ldots (40)$$

In general, loss in head due to friction for a steady flow in straight circular pipes is given by the formula:

$$h_f = K\,L\,d^{n-3}\,v^n\,\nu^{2-n} \quad \ldots\ldots (41)$$

and for two different liquids when the flow is turbulent,

$$\frac{h_{fw}}{h_{f0}} = \frac{v_w^{\;n}\,\nu_w^{\;2-n}}{v_0^{\;n}\,\nu_0^{\;2-n}}$$

Since v is a function of G:

$$\frac{h_{fw}}{h_{f0}} = \frac{G_w^{\;n}\,\nu_w^{\;2-n}}{G_0^{\;n}\,\nu_0^{\;2-n}} \quad \ldots\ldots (42)$$

Substituting the value of h_{f0} from Equation (42) in Equation (40):

$$h_0 = h_w + h_{fw}\left(1 - \frac{G_0^{\;n}\,\nu_0^{\;2-n}}{G_w^{\;n}\,\nu_w^{\;2-n}}\right) \quad \ldots\ldots (43)$$

Experiments* indicate that for a curved rotating channel, the critical velocity is not proportional to the value of the coefficient of viscosity; that is, its value does not conform to the theoretical condition that $\frac{v\,d}{\nu}$ is constant.

Moreover, to solve Equation (43), it is necessary to know the laws of frictional resistance for revolving channels in liquids of different viscosity and the effect of viscosity on sudden enlargement losses. As far as the writer knows, this information is not available at present.

Argument No. 2.—In the model of a pump, the dimensions of which are truly proportional to the original, G is proportional to $u\,b\,d$, when u is the peripheral velocity, d is the diameter of the impeller, and b is the width of the impeller.

Since, when N equals revolutions per minute,

$$u = \frac{\pi\,d\,N}{60} \quad \ldots\ldots (44)$$

and b is proportional to $N\,b\,d^2$. In the same manner, h is proportional to $N^2\,d^2$.

For two geometrically similar pumps:

$$\frac{G_1}{G_2} = \frac{N_1\,d_1^{\;2}\,b_1}{N_2\,d_2^{\;2}\,b_2} \quad \ldots\ldots (45)$$

and,

$$\frac{h_1}{h_2} = \frac{N_1^{\;2}\,d_1^{\;2}}{N_2\,d_2} \quad \ldots\ldots (46)$$

If the ratio of all the linear dimensions between the given pumps is M, then from Equations (45) and (46):

$$G_1 = G_2\,M^3\,\frac{N_1}{N_2} \quad \ldots\ldots (47)$$

$$h_1 = h_2\,M^2\,\frac{N_1^{\;2}}{N_2^{\;2}} \quad \ldots\ldots (48)$$

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* Staunton, "Friction".

and (correspondingly),

$$B\,H\,P_1 = B\,H\,P_2\,M^5\,\frac{N_1^{\,3}}{N_2^{\,3}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (49)$$

These equations based on dynamic similarity are found to be approximately correct.

For instance with ratios of 1 : 2, the specific speed of pumps changed 7%, although theoretically to satisfy the dynamic similarity the specific speed should have been constant.

The viscosities of oils being moved by centrifugal pumps reach a value of 6 000 S. s. u. (Saybolt seconds universal) for larger pumps and values as high as 3 000 S. s. u. are very common.

With corresponding ratio of kinematic viscosities of approximately,

$$\frac{\nu_0}{\nu_w} = 1\,295 \text{ and } 648$$

For $\frac{\nu_0}{\nu_w} = 18$, which was given by Professor O'Brien as most common in practice, the head capacity is practically the same for oil and water for pumps 3 in. in size and greater. The only change which takes place is in horse-power which increases slightly due to increased disk friction, depending on the diameter of the impeller and the operating speed. Consequently, little new information would be gained by testing pumps within the range of viscosities mentioned by Professor O'Brien.

For higher values of viscosities the method of testing pumps suggested by Professor O'Brien does not seem to be practicable, because of the "scale effect" and due to reasons outlined in Argument No. 1.

Professor O'Brien asks how the oil-pumping characteristics predicted from water characteristics given in the paper compare with the actual oil-pumping characteristics, as obtained by experiment. The agreement with the actual tests was very close.

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Paper No. 1735

COUNTY SEWER DISTRICT WORK IN OHIO AND ASSESSMENT OF COST ACCORDING TO BENEFITS*

BY E. G. BRADBURY,† M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. O. BONNEY, R. F. MACDOWELL, F. D. STEWART, W. M. OLSON, FRANK C. TOLLES, AND E. G. BRADBURY.

SYNOPSIS

Under Ohio laws, county sewer districts may be created and sewers and water mains may be constructed therein, the cost being defrayed by assessments levied on property benefited. More than 2 000 miles of construction have been executed under these provisions, largely in suburban or rural areas.

The character of the territory served involves special problems of design both as to boundaries of districts and provision for future development. The writer's basis of estimating proper sewer and water main capacities is described.

In the plan for the apportionment of assessments for trunk sewers in districts only partly sewered, the fundamental basis is the enhancement of value of the property affected, by reason of the improvement. The actual effect on market price cannot be predicted with accuracy, but a theoretical enhancement is worked out. The factors used are:

1.—The future cost to any part of the district for additional trunk or main sewers, for which a present credit is allowed.
2.—The estimated period in the future when service will be given to each part of the district, and an adjustment on present worth basis.
3.—The relative value of the land, a factor being used proportionate to the square roots of appraised values.

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* Presented at the meeting of the Sanitary Engineering Division, Columbus, Ohio, October 13, 1927.

† County San. Engr., Columbus, Ohio.

Similar methods are used in case of storm sewers and water mains, subject to special considerations due to the character of each.

History of Legislation

For a number of years the County Sewer District Law of Ohio has been a unique piece of legislation. Until recently, if the writer is correctly informed, it has represented the only attempt to provide for sewer and water facilities in unincorporated suburban and rural communities by laws of general application. One or two other States, however, now (1927) have laws of somewhat similar character.

This law is not to be confused with the "Sanitary District Act of Ohio," which authorizes the creation by the Court of Common Pleas of Sanitary Districts, including two or more political subdivisions, and the construction therein of main sewerage and water supply works for the service of the several units of such District, as the latter Act is designed for the service of contiguous or near-by municipalities, and makes no provision for local service.

The Ohio County Sewer District Law* dates back to 1911, when a crude and insufficient enactment sought to provide for the creation of county sewer districts and the construction of sewers therein. This law was soon found to be faulty and was amended in 1913 to a workable form. Some years later, further sections were added, authorizing the construction of water supplies in sewer districts and additional amendments have been passed from time to time.

An apparently radical defect in these laws, in that they did not provide for proper notice and hearings prior to the construction of improvements, was developed by litigation in Logan and Trumbull Counties within the past two years, the Courts in both cases indicating doubt as to the propriety of this procedure although failing actually to deny the constitutionality of the sections criticized. Amendments passed by the last session of the General Assembly provide in detail for such notice and hearings and the statute is now believed to be sound.

The Existing Law

As the law now stands, power is given to the county commissioners of any county to create, by resolution, sewer districts outside of municipalities, and to construct therein sewers, sewage treatment works, water supplies, and water mains, assessing the whole, or a part, of the cost on the property benefited. By authority of the council of any city or village, a part, or the whole, of such municipality may be included as a part, or the whole, of a county sewer district. Provision is made for joint construction and maintenance by two or more political subdivisions. The construction of utilities of this class by private interests, without the approval of the county authorities, is forbidden. The appointment of an engineer and assistants is provided for.

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* Comprised in Sections 6602–1 to 6602–33 of the Revised Statutes of Ohio.

The details of procedure are described at considerable length. A general plan of sewerage or water supply for the entire area is the first requirement after the creation of a district. Then follow detailed plans, specifications, estimates, and tentative assessments for such parts as it may be necessary to construct, a resolution of necessity, notices, public hearing, ratification or amendment, and the resolution to proceed. Provision is made for the issuance of bonds or certificates of indebtedness and the levying of revised assessments based on actual cost, but substantially in the same proportion as the tentative assessment.

RESULTING SANITARY DEVELOPMENT

Twenty-six of the eighty-eight counties of Ohio have acted under this law, in its various forms, and the extent and value of the work have far exceeded expectations. In an effort to secure full information, a letter of inquiry was mailed to the Sanitary Engineer of each of these counties. In the twelve counties from which replies were received there have been created seventy-nine sewer districts, with a total area of 336 600 acres. In these districts, sewers constructed or under construction total 807 miles, and water mains, 797 miles. Fifteen sewage disposal plants and six water supplies are reported in the same counties. Data from the remaining counties are not available, but it is very conservative to assume that more than 2 000 miles of sewers and water mains have been built under this procedure within the State.

Most of the work is the extension of the sewer and water systems of the larger municipalities, although quite a number of summer resorts and small rural communities have been provided with independent systems. The possibility of competent and systematic control of these matters in suburban areas is of great advantage to the cities, as inadequate design and improper construction are avoided, development is encouraged, and utilities are constructed at the entire expense of the property benefited. The city, of course, must be compensated for services in transporting and disposing of sewage and for water furnished.

In some of the counties, a city annexing territory that has been improved under this law simply takes over the sewers and water mains therein without compensation, and continues their operation. In others, a plan has been developed by which the city pays present value of these improvements when annexations are made. This is a question to be studied in each case, as the proper solution depends on how the city finances its own work of the same character. It is often a complex and many-sided problem.

The collection of interest on deferred installments of assessments at a rate equal to that on bonds sold to meet cost of construction, and a provision of the law that maintenance assessments may be levied as necessary, make possible the operation of sewer and water systems for rural communities much smaller than can be provided for by any other method. One water supply in Franklin County, consisting of a deep well, pumps, an elevated tank, and a distribution system, was operated for several months with twelve service taps and has never had more than seventy-five customers, yet water is sold at 35 cents per 1 000 gal., plus a service charge of 50 cents per month, and after four years of operation the original maintenance fund is intact and a small surplus has been accumulated. This is possible because there is no interest charge included in the water rates and a large replacement fund need not be built up, as future contingencies can be met when they arise by further assessment. This project was installed at a cost to the property owners of about $100 per lot.

Extent of Districts and Sewage Flow

The determination of the proper boundaries of a sewer district presents a rather difficult problem. Correct sewer design requires that provision be made for many years in the future, and, within the time designed for, the natural growth of the community will cover much land which is now used only for agricultural purposes, although usually valued at speculative prices. Often such territory, if not included, can not be later sewered except at great expense. Many owners resent even a small present assessment, and demand that their property be omitted. When such objection becomes general over an outlying part of a district, and cannot be satisfied by explanation, it is impossible to proceed against it, and a more short-sighted policy must be adopted.

In many of the districts there is much entirely undeveloped territory. This condition renders design somewhat difficult, because of the problematical character of future development. It is reasonable to expect such areas to become very largely residential in character, although more or less industrial activity will probably follow railroad lines; ordinarily, commercial development will be limited to small local trading centers. In Franklin County the practice is to design for residential purposes only, using figures sufficiently liberal to take care of any reasonably probable manufactural or commercial demand.

The basis of sewer design used by the writer differs somewhat from the ordinary standards. It recognizes the probability that there may be a congestion of population in any given part of the district and the possibility that this may occur anywhere in the district, but is not likely to extend over large areas. It provides for a wider variation between average and maximum flows in small areas than in large; and it anticipates a greater ground-water infiltration than is assumed by many engineers. Although using the most careful methods of construction, the writer has never been able to prevent material increases of flow during wet periods, due to infiltration through joints and manholes, admission of surface water through manhole tops, leakage of house connections, and doubtless some water from roofs or surface water inlets surreptitiously connected. It is of some interest to note that, with manholes 300 ft. apart, an average of one leak in each manhole equivalent to a ¼-in. orifice under 2-ft. head, will amount to about 25 000 gal. daily per mile, or 750 gal. daily per acre. An average of 2 drops per sec. from each joint of a pipe sewer will total about the same figures. That is, if the leakage is to be kept down to 750 gal. per acre, the infiltration for each 300 ft. of sewer must not exceed one ¼-in. leak, or a somewhat rapid dripping from each joint.

On the other hand, the average domestic consumption in Columbus is about 50 gal. per capita daily, or approximately 50% of the average per capita use for all purposes. This figure is accurately verified by the use of water in residential suburbs controlled by master meters.

BASIS OF DESIGN

In view of these considerations the following basis of sewer design has been established: (1) The future population is estimated according to a curve ranging from 40 people per acre for areas of 100 acres or less to 20 people per acre for areas of 500 acres or more; (2) it is assumed that the domestic sewage will average 75 gal. per capita daily; (3) that the maximum domestic flow will vary along a curve from 300% of the average in areas of 100 acres or less to 225% in areas of 1 000 acres or more; and (4) for ground-water infiltration, a flat rate of 3 000 gal. per acre daily is assumed. The net result of these figures is as follows:

Acres.	Cubic feet per second per acre.
50 or less	0.020
51 to 100	0.019
101 to 125	0.018
126 to 150	0.017
151 to 175	0.016
176 to 200	0.015
201 to 250	0.014
251 to 300	0.013
301 to 400	0.012
401 to 500	0.011
501 or more	0.010

In applying this tabulation quantities corresponding to the total area contributing are used and not a cumulative total. It will be noted that larger capacities are provided for small areas than are advocated by many engineers. This seems desirable in view of the fact that overtaxed sewers are much more frequently found near the upper end of a system than in the intercepting or outfall sewers. In the larger areas the capacity is approximately equal to 320 gal. per capita daily for a population of 20 per acre.

In designing general water systems the same uncertainty as to the future exists, and the engineer must avoid the extremes of hopeless inadequacy and dangerous optimism. Such a layout as will doubtless ultimately be required would involve capacities greater than are likely to be needed for many years, with consequent unduly heavy present assessments; it is also necessary to avoid the danger of insufficiency of service in too short a period. The possibility of future reinforcing mains and the character of the fire risk are taken into consideration in the Franklin County Department, and the general rule is to design for 20 people per acre, using an average of 50 gal. per capita, with a maximum of 225% of the average, plus a fire provision equal to three fire streams for the first 500 acres, plus one stream for each additional 500 acres. In view of the residential character of the expected development, this should provide satisfactory service for a reasonable period of gradual growth. In case of greater demand, supplementary mains will have to be constructed later.

Assessments

Perhaps the most interesting feature of the work is the apportionment of assessments for trunk sewers and the larger water mains. The law provides that, where such works are constructed, the abutting property shall be assessed for local service, and the excess cost, less such part as may be paid by the county at large, shall be assessed over the entire area benefited, including the abutting property, as a district assessment divided in proportion to benefits.

The first question arising from this provision is as to the amount, if any, that should be paid by the county, and included in general taxes. In municipal work, it is frequently provided by law that some part of sewer and water systems be financed by such taxes levied over the entire city. J. L. Van Ornum, M. Am. Soc. C. E.,* found that 19 out of 50 cities investigated paid the cost of main sewers by general taxation, and that 6 more paid the cost less a local assessment on abutting property. He states that this is proper in view of the general benefit to the community due to the establishment of a proper sewer system. In a report to the Sewerage Commissioners of Brockton, Mass., published in 1894, F. Herbert Snow, M. Am. Soc. C. E., stated that the Massachusetts law required that from one-fourth to two-thirds of the cost of common sewers should be paid from tax levies, and that, in his judgment, in the case of a separate system, one-fourth of the total cost of the system fairly represented the share of benefit accruing to the city as a whole.

There is, however, an essential difference between municipal and county conditions, in that all parts of a city may reasonably be expected to require sewers and water mains, either at once or in the very near future, while counties include, as a rule, large areas of agricultural land so remote from any city as to remove any possibility of urban development for generations. A general county tax must include these farm lands and, except in rare cases and under very unusual conditions, the payment of any part of the cost of this work by such taxation is unjust and cannot be considered. If the law permitted the levying of a tax within the limits of a sewer district, it would be a very different matter, but as this is not the case and since it is necessary to consider each sewer district as a separate body politic in designing and financing these improvements, the cost of each system must be defrayed by the assessment method.

When the law, as in this case, requires apportionment in proportion to benefits, it is almost universally held that the judgment of the assessing board will not be disturbed by the Courts unless fraud is shown or confiscatory assessments levied. This principle, together with other rules affecting assessments, has been very clearly stated in a recent Ohio decision.†

Assessment Proportional to Enhancement

The writer is of the opinion that enhancement of value is the proper and logical, as well as the legal, measure of benefit, and that the use made of the property at the time should not be considered, except as it may affect the major question. An assessment once levied cannot be changed, and, if he chooses, the owner of vacant land may construct buildings soon after the assessment is levied, that will make greater use of the sewer than those previously built by his neighbors. Failure of any owner to utilize his land should not relieve him of his proper charge. These principles are supported by the best legal authorities and by numerous Courts.*

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* "Theory and Practice of Special Assessments," *Transactions*, Am. Soc. C. E., Vol. XXXVIII (1897), p. 336.

† Rogers *vs.* Johnson, Treas., 21 Ohio App. May 7, 1926, Mauck, P. J.

The enhancement basis being adopted, it is evident that the character and value of the land become factors in estimating benefit. It has been held by the Courts that either valuation or area, or both, may constitute a proper basis for such apportionment. Mr. Snow gives an illustration showing that the valuation basis, if used in direct proportion, must work a hardship on the more valuable property, but it may be safely assumed that land worth $2 000 per acre must appreciate in value (expressed in dollars per acre) to a greater extent than $200 land, when furnished with sewer and water facilities. Assuming that the cost of these improvements was equal to an average of $200 per acre, it cannot be true that the one is benefited only 10%, while the other is doubled in value. The fact doubtless lies somewhere between the two extremes and after much consideration the writer has reached the conclusion that if, in connection with the other factors later described, the assessment is apportioned in the ratio of the square root of the values, substantial justice will be done. This brings about a result approximately equivalent to spreading one-half the assessment in the ratio of value and one-half on a direct area basis. Land values only are considered, as the buildings are under control of the owner and may be entirely changed at any time.

It is admittedly impossible to determine the precise effect of any public improvement on the market value of property in the district. All that can be done is to estimate what should be the effect. All authorities agree that it is impossible to apportion an assessment with absolute accuracy and correctness, but an effort should be made to establish a method which will result in substantial equity.

As far as the writer has been able to learn, no attempt has ever been made to analyze this problem, although Professor Van Ornum refers to a case in which various factors were considered, but without explaining the method used. Many engineers have adopted a more or less arbitrary plan of establishing zones at varying distances from the improvement and fixing percentages representing the relation of the rates in such zones. In many cases, the assessment is apportioned simply in accordance with the personal judgment of the officials as to relative benefits. The plan originated by the writer, which is used in Franklin County, and has been adopted by several other counties in Ohio, represents an effort to establish a fair and equitable basis and, as it is believed to present some new ideas, will be described in detail.

METHOD OF APPORTIONMENT

The plan is based on the following premises:

1.—The measure of benefit is the difference in value of the property before and after the improvement is made.

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* City of Butte 2, School Dist. No. 1, 29 Mont. 336 339, 74 Pac. 869 (1904). See, also, "Taxation by Special Assessments," by Page and Jones; "Law of Special Assessments," by Hamilton; and "Municipal Corporations," by McQuillan.

2.—Valuable land is benefited more than cheap land, although not in proportion to the ratio of the values, the square root of the value ratio being assumed to represent this item fairly.

3.—The present use of the land, or the presence or absence of buildings is not ordinarily a factor, being subject to radical change and wholly under control of the owner.

4.—In the case of a district completely sewered at one operation, the cost of trunk sewers and disposal facilities could fairly be assessed over the district on an area basis, subject only to the valuation correction, and such an apportionment should represent a reasonable estimate of the enhancement of value.

5.—When only the main works of the system are built, designed to serve the whole district ultimately, the assessment should be so adjusted that when the system is completed, each property in the district will have been assessed equally per unit of area, as of the date when service is given, subject to correction for valuation.

6.—Following the thought of Premise 5, it is evident that property which will later be assessed for a share of main sewer extensions or branch mains, is correspondingly less benefited by the sewer now built than property which will not require such an extension or branch. This feature of the problem may be called "accessibility".

7.—It is equally clear that property which is not given immediate service is less benefited than that which has such service, and that a fair measure of this factor, which may be called "time", is the ratio of the present worth of a dollar, for a period equal to the probable time which will elapse before service is rendered, to the basic dollar.

In order to develop such a distribution of the district assessment it is necessary to estimate the cost of future extensions and branch mains, the period at which each portion of the district will be so developed as to require sewerage, and the value of the land. The first is a simple matter of estimating costs from the general plan. The second is obtained by dividing the district map into rather small zones, calling in the services of several qualified real estate experts familiar with the district, securing the estimate of each as to the reasonable development period for each zone, and striking an average representing their combined judgment. The third item was formerly handled in a similar manner, but an official re-appraisal of all property in the county by a force of competent appraisers has made this unnecessary.

Computation of Assessments

The detailed procedure in distributing the district assessment for a trunk sewer on these principles is as follows:

1.—*Present and Future Excess Cost.*—Estimate the total present and future excess cost of all trunk and branch sewers of the system above the part of such cost which will be assessed on abutting property for local service.

2.—*Sub-Districts.*—Divide the district into sub-districts (for purposes of assessment only), each of which includes the whole area served by an extension or branch larger than 8 in., or which contributes either directly or by 8-in. laterals to the main trunk.

3.—*Time Zones.*—Divide these sub-districts into the several time zones representing the period when service is expected to be provided, such as "immediate", "2 years", "5 years", etc.

4.—Accessibility.—Apportion the total present and future excess cost to the several sub-districts in proportion to area, and deduct from the amount charged to each sub-district its own estimated excess future extension or branch main cost.

5.—Time.—Divide the amount thus apportioned to each sub-district among the several time zones therein and multiply the share of each zone by the present worth of one dollar for the period assumed for that zone. Add the results and determine the percentage relation of each to their sum. Apportion the total present district assessment over the zones in the ratio thus established.

6.—Valuation.—It is not practicable to correct each individual property for valuation. Areas of approximately equal unit value are laid out and a fair average is used for each. The apportionment for each area is then computed from the accessibility and time results and multiplied by a factor equal to the square root of the quotient of the valuation per acre of that area divided by the lowest valuation per acre of any area in the district, or,

$$F = \sqrt{\frac{V}{V_{\text{min.}}}} \qquad \qquad (1)$$

in which, F is the factor applied to any area, A; V is the valuation per acre in A; and $V_{\text{min.}}$ is the lowest valuation per acre of any area within the district. The percentage relation of each result to their sum is determined, and this percentage, applied to the total amount of the present district assessment, gives the amount to be levied on each valuation area. From this, the rate per acre is readily determined and the apportionment for each property is a simple matter.

In most districts there are both platted and unplatted areas. In order to avoid discrimination against the latter, a reduction of 15 to 20% is made in all unplatted areas to allow for future streets. In platted territory, after fixing the total amount to be collected from each valuation area, this amount is distributed, using frontage, or more correctly, average lot width, as a general guide, this usually appearing to represent an equitable division. In some unusual cases, a combination of frontage and area has been used, but, with ordinary regular platting, this is unnecessary. Occasionally, some local condition requires special consideration; but, in general, assessments can be computed by the method described with satisfactory results.

These computations are not as difficult or complex as they seem. In the case of large districts they require a considerable amount of time and effort, but if they result in a fair and equitable solution of the problem, it is worth while. It may be mentioned that in the practical application of this method, it has been found that the assessments levied against outlying property not now given actual service, approximate reasonably the cost of the additional capacity provided to meet their future needs.

TYPICAL COMPUTATIONS

The process may be made more clear by the following example of an actual assessment in Franklin County, Ohio, covering one of the smaller projects. It will be seen that it results in placing the weight of the assessment on the valuable and immediate property and reducing the burden on the cheap and distant land to a small figure.

Computation of Assessment Rates: Marion Road Sewer.—

Amount to be assessed....................	$11 607.63
Total present and future excess cost...........	$14 338.26
Total area to be assessed.....................	622.85 acres

TABLE 1.—ACCESSIBILITY.

Sub-district. (1)	Acres. (2)	Average rate per acre. (3)	Preliminary apportionment. (4)	Future excess. (5)	Adjusted preliminary apportionment. (6)
A	240.47	$23.02	$5 535.72	$1 785.33	$3 750.39
B	84.12	23.02	1 936.48	224.10	1 712.38
C	60.79	23.02	1 399.41	250.80	1 148.61
D	38.10	23.02	877.08	268.80	608.28
E	61.51	23.02	1 415.98	201.60	1 214.38
F	137.86	23.02	3 173.59	0.00	3 173.59
Total.......	622.85	......	$14 338.26	$2 730.63	$11 607.63

TABLE 2.—TIME.

Zone. (1)	Time, in years. (2)	Zone area, in acres. (3)	Sub-district area, in acres. (4)	Adjusted preliminary apportionment for sub-district. (5)	Present worth of one dollar. (6)	Present worth. (7)	Percentage of total. (8)	Time adjustment of excess cost. (9)
A 1	5	26.05	240.47	$3 750.39	$0.747	$303.49	4.13	$479.40
A 2	10	49.80	240.47	3 750.39	0.558	433.39	5.90	684.85
A 3	15	71.08	240.47	3 750.39	0.417	462.27	6.29	730.12
A 4	20	98.54	240.47	3 750.39	0.312	455.16	0.20	719.67
B 1	5	4.68	84.12	1 712.38	0.747	71.16	0.97	112.59
B 2	8	3.57	84.12	1 712.38	0.627	45.56	0.62	71.97
B 3	15	43.21	84.12	1 712.38	0.417	366.79	4.99	579.22
B 4	20	32.66	84.12	1 712.38	0.312	207.42	2.82	327.34
C 1	8	3.57	60.79	1 148.61	0.627	42.29	0.58	67.32
C 2	15	42.19	60.79	1 148.61	0.417	332.41	4.52	524.66
C 3	20	15.03	60.79	1 148.61	0.312	88.60	1.21	140.45
D 1	3	14.89	38.10	608.28	0.84	199.69	2.72	315.73
D 2	7	23.21	38.10	608.28	0.665	246.42	3.36	390.02
E 1	3	21.95	61.51	1 214.38	0.84	364.02	4.96	575.74
E 2	5	20.34	61.51	1 214.38	0.747	299.97	4.08	473.59
E 3	7	19.22	61.51	1 214.38	0.665	222.34	3.44	399.30
F	Immediate	137.86	137.86	3 173.59	1.00	3 173.59	43.21	5 015.66
Totals....	....	622.85	....	....	....	$7 344.57	100.00	$11 607.63

EXPLANATION OF TABLES

Table 1 contains the computations to determine the preliminary apportionment of costs on the basis of accessibility. The average rate per acre (present and future), is found to be $\frac{14\,338.26}{622.85} = 23.02$. The values in Column (4) are the products of Columns (2) and (3), and show the amount which would have been chargeable to each sub-district (subject to correction for valuation) if the complete system had been constructed at this time. Column (5) shows the estimated excess cost of the future mains in each sub-district. Sub-District F is the area of present service. Column (6) is compiled by subtracting Column (5) from Column (4) for each sub-district, crediting each sub-district with its future expense.

The effect of time in the adjustment of excess costs is computed as shown in Table 2. Column (1) designates the division of Sub-Districts A, B, C, etc., into zones that will feel the benefits of the improvement in various estimated periods of time. The corresponding time intervals are listed in Column (2). Column (3) lists the area of each zone. Columns (4) and (5) correspond to Columns (2) and (6) in Table 1. Column (6) gives the present worth of \$1.00 at the end of the time interval indicated. Column (3) divided by Column (4) and the result multiplied by Column (5) gives the adjusted preliminary apportionment for each zone, and this multiplied by Column (6) gives the results shown in Column (7), or,

$$\text{Column (7)} = \frac{\text{Column (3)}}{\text{Column (4)}} \times \text{Column (5)} \times \text{Column (6)}$$

The total of Column (7) is, of course, less than the amount to be assessed and it is, therefore, proportionally expanded to the correct amount as indicated in Columns (8) and (9).

TABLE 3.—VALUATION.

Area. (1)	Value, per acre, (V). (2)	Area, in acres. (3)	Total area of time zone, in acres. (4)	Adjustment of excess cost for time zone. (5)	Value of $F = \sqrt{\frac{V}{200}}$ (6)	Time assessment × F. (7)	Percentage of total. (8)	Value adjustment for excess cost. (9)
A1(*a*)	\$ 500	23.13	26.05	\$ 479.40	\$1.581	\$ 672.97	2.92	\$ 338.93
A1(*b*	750	2.92	26.05	479.40	1.936	104.03	0.45	52.23
A2	300	49.80	49.80	684.85	1.225	838.94	3.64	422.52
A3	300	71.08	71.08	730.12	1.225	894.40	3.88	450.38
A4	200	93.54	93.54	719.67	1.	719.67	3.12	362.16
B1	500	4.68	4.68	112.59	1.581	178.00	0.77	89.38
B2	800	3.57	3.57	71.97	2.	143.94	0.62	71.97
B3	300	43.21	43.21	579.22	1.225	709.54	3.08	857.51
B4	200	32.66	32.66	327.34	1.	327.34	1.42	164.83
C1	800	3.57	3.57	67.32	2.	134.64	0.58	67.32
C2(*a*)	300	31.31	42.19	524.66	1.225	476.96	2.07	240.28
C2(*b*)	400	8.16	42.19	524.66	1.414	143.48	0.62	71.97
C2(*c*)	800	2.72	42.19	524.66	2.	67.64	0.29	33.66
C3	200	15.03	15.03	140.45	1.	140.45	0.61	70.81
D1	750	14.89	14.89	315.73	1.936	611.25	2.65	307.60
D2	400	23.21	23.21	390.02	1.414	551.49	2.39	277.42
E1	750	21.95	21.95	575.74	1.936	1 114.63	4.83	560.65
E2	600	20.34	20.34	473.59	1.732	820.26	3.56	413.23
E3	400	19.22	19.22	399.30	1.414	564.61	2.45	284.39
F1	1 000	27.00	137.86	5 015.66	2.236	2 196.46	9.52	1 105.05
F2	1 250	25.24	137.86	5 015.66	2.5	2 295.72	9.95	1 154.96
F3	1 500	20.04	137.86	5 015.66	2.739	1 997.01	8.66	1 005.22
F4	1 750	24.32	137.86	5 015.66	2.958	2 617.29	11.35	1 317.47
F5	2 000	41.26	137.86	5 015.66	3.162	4 746.58	20.57	2 387.69
Totals	....	622.85	....	....	....	\$23 067.30	100.00	\$11 607.63

Finally, the computations for the determination of the valuation correction are given in Table 3. It will be noted that some of the zones in Column (1) have been further subdivided in order to allow for a difference in valuation. In Column (2) are listed the values of the various areas per acre. The acreage of each valuation area is given in Column (3) and, for convenience in computation, Column (3), Table 2, is repeated as Column (4). Column (5) is a duplication of Column (9), Table 2. In Column (6) are listed the values of the factor, F, which equals $\sqrt{\frac{\text{Column (2)}}{200}}$. Column (3) divided by Column (4) and the result multiplied by Column (5) gives the time assessment for each valuation area and this multiplied by Column (6) makes a preliminary correction for value, shown in Column (7), or,

$$\text{Column (7)} = \frac{\text{Column (3)}}{\text{Column (4)}} \times \text{Column (5)} \times \text{Column (6)}$$

The total of Column (7) being greater than the amount to be collected, the items are reduced proportionally as indicated in Columns (8) and (9). Column (9) of Table 3 gives the amounts to be assessed in each area, and all that remains is to distribute them over the individual properties within the areas, the rate per acre being fixed by dividing each amount by the number of acres in its particular area, or,

$$\text{Rate per acre} = \frac{\text{Column (9)}}{\text{Column (3)}}$$

Storm Sewer Assessments

In the case of storm sewers special consideration must be given to land relieved of overflow conditions or otherwise benefited to a greater degree than other property, but if this feature is taken care of in a manner similar to local service cost as described in the detailed procedure previously outlined, the same methods can be thereafter used.

Assessments for Water Mains

Water mains present a different problem in one regard, that is, that the boundaries of the district to be served are not fixed as in the case of a sewer. It has been found that the only practicable and satisfactory method is to lay out a definite area on both sides of each main, covering a reasonable zone of influence with reference to other present or future mains, and ignore the inevitable circulation and overlapping of service through the several feeders, assuming the benefit of each to extend to a line between such feeders at a distance fairly representing the proportionate capacity. This being determined, the regular method may be used.

Conclusion

The system described has been in use for several years. In general, it has seemed to meet with public approval, and no difficulty has been found in explaining it in simple language to the average property owner. Many expressions of approval of the principles involved have been received from such owners as well as from attorneys and engineers. It is presented for consideration and discussion as an attempt to solve, as far as may be, a problem which has apparently heretofore received less attention than its importance merits.

DISCUSSION

O. BONNEY,* ASSOC. M. AM. SOC. C. E.—In connection with this paper it might be of interest if some of the points mentioned were discussed briefly from the standpoint of a representative of a municipality, of approximately 300 000 population, engaged in furnishing sewerage and water service to several such county sewer districts.

Recent estimates by the U. S. Census Bureau credit Columbus, Ohio, with a population of 291 000 as of July 1, 1927, the area of the city being approximately 32.5 sq. miles. The trend of development in and around the city is principally north, northwest, and east, which makes it necessary for sewage from practically all the Franklin County Sewer Districts to be transported and disposed of by the sewers and sewage disposal works of Columbus. Plans by the City, for intercepting sewers, relief sewers, and sewage treatment, contemplate provision of capacity for sewage from about 59 sq. miles outside the present city, a large part of this area being in established county sewer districts. To date, about 78 miles of sewers have been constructed under the jurisdiction of the County Sanitary Engineer, of which, about 51 miles are tributary to the sewerage system of the City of Columbus.

The City charges the population, residing in county sewer districts and in suburban villages adjacent thereto, tributary to its sewerage system, an amount which is arrived at in accordance with the following rate schedule: $4.00 per year for single houses; $3.50 per year for each side of a double house; $3.00 per year for each apartment of an apartment house; and $1.00 per employee per year in the case of stores or factories, the number of employees being based on the average number employed during a six months' period.

There are now (1927) approximately 10 000 persons in county sewer districts and in suburban villages tributary to the sewerage system of the city, who are paying annual charges to the City in accordance with these rates.

The rates now in use were established several years ago and were based on approximate estimates, and some officials have raised the question as to whether or not they do represent an adequate return to the city as measured by the money invested and the service rendered. The City is now making a comprehensive study of the situation with a view toward the establishment of a rate for transportation, pumping, and treating sewage from county sewer districts and suburban villages, which at once will be fair and equitable to all parties concerned and which will be based on the cost of construction, operation, and maintenance. This problem is being approached in accordance with established valuation and rate-making principles.

Close co-operation is maintained between the County and the City with respect to basic design assumptions, boundaries of districts, etc., and while the rates of sewage flow, ground-water infiltration, etc., provided for are not identically the same, results—which to all intents and purposes are in close agreement—are reached. In the matter of the allowance for ground-water infiltration there is considerable difference between the rates used by the

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* Sewerage Relief Engr., City of Columbus, Columbus, Ohio.

County and the City. Mr. Bradbury has stated that he provides capacity for 3 000 gal. per acre per day, while 750 gal. per acre per day—equivalent to about 25 000 or 30 000 gal. per mile of sewer per day—is provided for by the City.

There is perhaps some question as to the allowance of 3 000 gal. per acre per day. This is a higher figure than is commonly used, but conditions in some districts contiguous to the city may justify it. For some time there has been a doubt in the speaker's mind as to the sufficiency of the City's allowance. In one or two instances there have been indications that ground-water leakage exceeded 750 gal. per acre per day. In some districts in the city served by separate sanitary sewers and storm drains, considerable overloading of the sanitary sewers takes place during wet weather. To a certain extent ground-water infiltration no doubt accounts for some of the overloading, but roof water connections and the entrance of surface water make it practically impossible even to estimate approximately the quantity. Efforts are being made by the City to disconnect the roof water and surface water connections, and once this is accomplished infiltration measurements can be made. On about 15 miles of sanitary sewers now (1927) under construction in the city every precaution is being taken to reduce ground-water infiltration. A type of precast asphalt gasket is being used in the joints, 3-ft. lengths of pipe are being installed, and special attention is being paid to manhole construction. When this work is completed and before house connections have been made to it, it is planned to attempt to measure the leakage during a wet season.

In regard to the assessment of cost in proportion to benefits, Mr. Bradbury has presented what might appear to be a complex method of measuring the benefits derived by the installation of sewers and has shown it to be very simple in operation. It seems fitting and proper that the element of value before and after the sewer has been installed should be considered, and his method of handling this factor would seem to be just and equitable to all. It has not been the City's custom to adjust assessments on the basis of valuation. Ordinarily, 98% of the cost of sanitary sewers is assessed against the property benefited—2% being borne by the city at large. The 98% is usually distributed over the district in proportion to frontage benefits. In the case of storm drains the practice on the part of the City has been somewhat varied. In some cases the property benefited has been assessed the entire cost minus 2% plus the cost of drains in street and alley intersections and, in some instances, the entire cost has been carried by the city at large.

It is contended by some that the installation of sewers and water mains in county sewer districts contiguous to a municipal corporation operates to the disadvantage of the municipality in that a district once provided with sewer and water facilities and other utilities is not particularly desirous of being annexed to the municipality. This may be true to some extent but usually the advantages to a community to be derived by annexation overbalance any possible disadvantages; hence, when the district does come into the city it represents an asset to it in the way of complete and well-designed and constructed water and sewerage systems. Certainly a municipality is fortunate in having the extensions of its sewerage and water systems into county territory made under capable engineering supervision. Annexations to the City of Columbus since 1920 total approximately 11 sq. miles, which would indicate to some extent at least that the city is continuing its former steady growth in area and that the construction of sewers and water mains under the County Sewer District Law has not been disadvantageous to the city.

R. F. MacDowell,* M. Am. Soc. C. E.—The Sewer District Law, as originally adopted by the Ohio Legislature in 1911, was a rather crude and unworkable tool; but it contained the basic elements of a most needed and useful instrument. There was required only the refining effects of protests, Court actions, the experiences of use, and several legislative amendments over a period of seventeen years, to make it what is now felt to be a practically impregnable act. It meets a real public need in connection with the development of property for residential purposes in the suburban areas surrounding the larger centers of population. Other States would do well to profit from Ohio's experience in the creation and operation of its Sewer District Law.

To show the extent to which the law is being utilized in Ohio, it was estimated, that when the law was in jeopardy for several months preceding the 1926-27 session of the Ohio Legislature, water and sewer improvements costing approximately $50 000 000 were under way or contemplated under the law in the twenty or more counties of the State which had established sewer districts. The development of the law and the use being made of it, are but a natural outcome of the tendency of urban residents to get out into the open spaces surrounding the cities proper when building their homes. Not only are the unincorporated areas surrounding the larger cities of Ohio taking advantage of the law, but many of the suburban villages are utilizing it to finance water and sewer improvements which their taxing limitations would otherwise prevent. The law has also been of especial service to summer resort communities, from which much typhoid previously emanated due to bad sanitary conditions. These resorts are being rapidly cleaned up and given modern conveniences by the help of the law.

The determination of the proper boundaries of a sewer district (or the area to be assessed for any improvement) is, as Mr. Bradbury states, a difficult one because it depends on judgment as to the probable development of the territory in the future. It would obviously be unjust to assess for a trunk sewer the area of an entire township (if such area happened to be on the water-shed of such trunk) when only a small part of the area is immediately benefited. On the other hand, the engineer must be firm in including, within an assessment district, farm land which is immediately adjacent to and in the line of subdivision development, and is, therefore, quite obviously benefited by the improvement in spite of most vociferous protests to the contrary by the present property owners who are thinking of benefits solely in terms of added income from the sale of crops. Under the Ohio Sewer District Law, the Courts will undoubtedly uphold any assessment if the engineer has been reasonable.

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* Civ. and San. Engr., Cleveland, Ohio.

As regards the basis of design for sewers, especially that of per capita flow, the speaker believes with Mr. Bradbury that it is necessary to be quite generous in making allowance for concentration of development and also for ground-water infiltration. The speaker finds the curves of maximum, as compared with average, flows in sewers—as prepared by Robert B. Morse, M. Am. Soc. C. E.—to be quite useful and safe for small sewers. The matter of ground-water infiltration is a serious one, and should always be given careful consideration, depending on the character of the soil and the normal ground-water level. Although bituminous materials have been used for sewer joints for many years, their general use has developed only recently. This has resulted in a flooding of the market with many bituminous compounds, some of which are almost worthless, and the engineer must be prepared to approve only those which will be effective. A reputable testing laboratory at Cleveland, in co-operation with the speaker, has prepared the following specifications for bituminous compounds and the speaker requires that any such compound proposed for use in sewers must meet these specifications:

"The compound shall consist of bituminous material mixed with mineral matter. It shall melt and flow freely at 250° Fahr., and shall show no loss in weight when brought to a temperature of 400° Fahr. It shall adhere firmly to glazed surfaces of pipes, sufficiently to withstand 10 lb. internal pressure under conditions obtaining in practice. When set it shall have sufficient elasticity to permit of slight lateral flexure without injury to the joints or breaking the adhesion to the pipes.

"The amount of bituminous material extracted by carbon bisulfide shall not be less than 40% and the weight of mineral matter determined as ash shall not exceed 50 per cent.

"It shall show no deterioration of any kind when immersed for a period of 5 days in a 1% solution of hydrochloric acid or in a 5% solution of caustic potash."

A pre-cast bituminous joint has been developed recently which the speaker believes to be a progressive step, although its efficiency depends to quite an extent on the skill and care with which it is pressed "home" to fill the joint. Until a sewer pipe joint has been developed which is "fool-proof" and certain to make a tight joint, under ordinary conditions of placing, ground-water infiltration will continue, and, in designing sewers, the engineer should make a generous allowance for this factor.

The major part of Mr. Bradbury's paper covers the subject of special assessments, and rightfully so, as Mr. Bradbury has spent a great deal of time and has developed much original thought on the subject. In reading much of the available literature on the subject, the speaker has been impressed with the fact, that, as Mr. Bradbury states, the problems involved in special assessments have never been thoroughly analyzed. At least, no hard-and-fast rules, and but few basic principles, seem to be generally accepted. There is a wide divergence of opinion as to what elements make up special benefits and various methods are, therefore, in force in different parts of the United States. This is probably due, in part, to the fact that variation is necessary in order to conform to local conditions as well as to type of improvement.

Quoting a well-expressed definition of special assessments, they are "compulsory contributions paid once and for all to defray the cost of a specific improvement to property, undertaken in the public interest, and levied by the Government in proportion to the special benefits accruing to the property owners". Another defines them as "general proportional contributions of wealth levied against land and collected from its owners and occupants to defray the costs of specified public improvements made, or of specified public service undertaken, in the interest of the general public."

Special assessments differ from general taxes in that the latter are levied to provide for all general public needs, whereas the former are levied to meet some special need of property situated in an area created for the express purpose of such levy and possessing no other purpose. Furthermore, taxes are levied continuously, while special assessments are exceptional, as to time as well as locality, and should be made only where the property assessed is actually increased in value to an extent at least equal to the amount of the assessment.

In one sense, almost all public improvements especially benefit some individual or a part of the people of any community. It thus becomes a matter of judgment how far to go in making special assessments. For example, the creation and maintenance of parks are financed in the City of St. Louis, Mo., by special assessment within definite park districts. Likewise, the City of Cleveland is now installing "white way" lighting systems, the cost of which is assessed on the property benefited thereby. Special assessments are also being used over the United States for financing irrigation, drainage, excess condemnation, re-paving, and rapid transit as well as water, sewers, sidewalks, paving, and grading improvements. Thus, it may be stated that the special assessment method of financing public improvements is gaining in favor. This is probably due to general tax limitations, and also to the fact that it is being more and more recognized that special assessments place the burden directly on the property benefited. The problem is in determining what property is benefited, and to what extent.

The relative amount of the assessment in each case rests with the engineer, or with a group of special appraisers, and although it is practically impossible for them to secure ideal justice for every one affected, results may be obtained by careful thought and intention to a few basic principles, which cannot be criticized and, therefore, will not be invalidated by the Courts.

Although it is fundamental that any property which is not especially benefited by an improvement should not be especially assessed, it is equally true that all property which receives some benefit should receive some part of the assessment. All public, quasi-public, or private property seems to be assessable when benefited, such as schools, cemeteries, churches, parks, and railroad rights of way. It seems to be recognized that any exemption of benefited property from assessment is unsound, and the assessment should be spread where the benefit rests, whether such benefit be to the general public or to a private individual, and any exemption from assessment places an added burden on the remainder of the property to be assessed.

Although the topographic and other natural features of the property are quite often made the cause of reductions of the special assessments, some Courts seem to hold that no variation in the unit assessment need be made on this account. One Court has decided, for example, that a lot which lies below the level of the street to be improved, does not benefit less than the adjoining lot at the street level, as the improvement is there, to be taken advantage of by filling the property, and the assessment for such lot, therefore, was upheld. Some authorities and also some Courts, however, disagree with this reasoning, stating that the assessment for such lot should be reduced to some extent.

There are also other reductions which should be made. For example, where allotted and unallotted lands are being assessed for the same improvement, a part of the latter should be exempted, equivalent to the average area which is in streets in the allotted section, and is thus free from assessment.

The power to levy special assessments is generally held by the Courts to be a continuous one; that is, as many assessments may be made on any parcel of land, for benefits derived from various extensions to any improvement, as may be necessary, so long as special benefits can be shown from each assessment. This principle may be taken advantage of, for example, in a progressive program of sewer construction extending over a term of years, to complete a sewer system with a single main outlet. The outlying territory to be ultimately drained by the main trunk sewer may be assessed for its reasonable proportion of the cost of each part of the outlet, interceptor, trunk, or lateral sewer until the local service pipe is provided. Such assessments, however, should always be made so that the entire area drained by the system will be assessed an approximately equal amount, as of the date when the local service is obtained.

Although there are many factors involved in the determination of proportional benefits, as shown by Mr. Bradbury, the speaker wishes particularly to mention the use of valuation. The author has developed a method for the use of valuation of land as a factor in determining the proper apportionment of assessments, which is quite unique and deserves recognition. There is no doubt in the speaker's opinion that valuation, if properly applied, leads in the direction of ideal justice in the computing of assessments. There are, however, many difficulties as well as objections to its use, which, for purposes of discussion, the speaker wishes to emphasize.

When valuation is used, the assessment is distributed in proportion to the assessed valuation of the land at the time the improvement is made, or some proportion of such valuation. Quoting from the *National Municipal Review* of February, 1922:

> "It favors cheap land at the expense of dear land and fails to recognize that the present value of land is due to conditions existing before the improvement is contemplated or completed and that these conditions may be completely altered by the improvement."

And, it may be added, by other conditions arising after the assessment is made. In other words, the valuation method, although in some cases particularly adaptable to the securing of equitable assessments in proportion to benefits, is unstable when compared with the area and the foot frontage, as these do not change from time to time, as does the valuation of property. Special assessments should always be levied so as to produce equal and uniform justice throughout the life of the improvement. It is for this reason that no consideration should be given to built-up as compared to vacant land, as the benefit is the same in each case.

The use of valuation may result in unjust assessment as the valuation of property may change rapidly, due to private improvements, whereas the benefit from the public improvements is nearly the same for improved as for unimproved property; also, valuations are often unreal and unjust and decidedly not up to date, if taken from the tax duplicate, or merely the present judgment of one or more men, in case real estate experts are called in to pass on the matter.

The author's line of reasoning as regards the use of valuation is quite logical but the speaker believes that valuation cannot be used with surety that justice is being served. The determination of relative valuations is arbitrary and assessments made with valuation as a factor will be likewise arbitrary. High-class property should not necessarily be assessed more than low class, merely because it is more valuable, because often the so-called low-class property is made high class merely by the addition of water and sewer improvements. It is agreed that the use made of land is not a measure of benefit, but surely it affects its value. Therefore, does it not follow that value in itself is not a measure of benefit? On the other hand, there is no doubt that enhancement of value by reason of improvements is a true measure of the special benefit to property, or that failure to use land should not relieve the owner from assessment. However, property enhances in value from improvements in proportion to its accessibility and the time in the future when use will be made of the improvement. If these factors, as well as the topography or usable area of the property, are properly taken into consideration, there is doubt as to whether the valuation factor as proposed by the author does not carry the decimal point too far, as it were; especially is this true when the relative valuation of the various parcels must be first assumed and then an arbitrary assumption made that the values vary in proportion to the square root.

F. D. STEWART,* ASSOC. M. AM. SOC. C. E.—The Ohio Statutes require the approval—by the State Department of Health—of all plans of public water supply and sewerage improvements within the State and, consequently, the Department is brought into close contact with the work of the various counties in installing these improvements. The speaker has been associated with that Department since about 1920 and has been in a position to see the operation of the County Sewer District Law expand from its original small proportions to its present status.

It has been noted by Mr. Bradbury that the bulk of the work being done by counties is in territory contiguous to the larger cities. The value of having sewerage and water improvements designed and installed in a logical and comprehensive manner and in conformity with the same utilities of the adjacent city cannot be over-estimated. To any one familiar with the difficulty of securing proper sanitation in suburban areas the advantages of such a central working authority, as this law provides, is apparent. Without proper sewerage and water supply the areas will abound with unsafe wells (either contaminated or subject to contamination), cesspools, septic tanks, and misused storm drains, with resultant nuisances. In counties well organized for this work, sanitary improvements now precede to a greater or less extent the actual occupation of the outlying areas of cities. It is true that the county officials, in establishing the boundaries of sewer districts and designing the improvements, must guard against being unduly influenced by the optimism of realtors, but if good sound judgment is exercised no real injustice in the way of assessments will occur. Furthermore, the law as it stands to-day provides ample recourse to any property owner who does not consider the action of the county right or just, either in the inclusion of his property in the district or in the tentative assessment.

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* Asst. Engr., State Dept. of Health, Columbus, Ohio.

The law which provides for the inclusion of parts of municipalities in county sewer districts has been found to be of great advantage. This permits the solution of sewerage problems in accordance with topography without limitations of corporate boundaries. In most instances this has been of greatest value to the cities in providing sewer outlets for territory within the city boundaries inaccessible to existing sewerage improvements.

There are in Ohio a number of summer resorts located along the shore of Lake Erie and around small inland bodies of water to which there is a large influx of people during the summer season. These resorts are generally remote from the centers of population and their sanitation as it relates to water supply and sewerage presents a most perplexing problem. The outdoor privy vault, formerly associated with summer vacationing, is now barely tolerated, and the demand for modern conveniences is insistent. It is here that the County Sewer District Law can function to a distinct advantage as the areas are usually unincorporated. In fact, it has been found in Ohio that in no other way can proper sewerage and water supply improvements be installed. The real obstacle in securing installation of improvements is the item of cost. The property is usually of lower value than city property. Also, the use of the adjacent bodies of water for bathing may render the disposal of sewage difficult and costly. The problem, therefore, calls for the exercise of the best engineering skill and economy of design and construction.

The existence of the County Sewer District Law often provides a means of securing sewerage and sewage disposal for small municipalities which are hampered by statutory debt and tax limitations. The municipal authorities may petition the county commissioners to include the municipality as a whole or part of a county sewer district, and thereupon the county may install the improvements and assess the cost directly on the benefited property.

The law vests with the county commissioners the authority to initiate work under the County Sewer District Law. Nevertheless, it has become a custom in the State to require a petition from a percentage, and usually in the case of local improvements, a majority, of property owners to be benefited by immediate installations. Occasionally, a board of county commissioners will refuse to act in this regard for areas obviously in need of improvements, even on receiving petitions from a large percentage of property owners. Relief from such a situation is afforded by the Ohio General Code. This provides that whenever formal complaint by constituted authorities is made with reference to unsanitary conditions in any area outside of corporate municipalities, the State Director of Health shall investigate the complaint in order to substantiate the claim. If conditions are found as alleged, the State Department of Health may order the establishment of a county sewer district by the county commissioners—to include the area concerning which complaint has been made—and the installation of sewerage therein.

The law further provides that on failure to comply with such an order of the State Department of Health, the Act may be enforced by a writ of mandamus issued by any Court authorized to issue such writs. Similar action may be taken with respect to water supply improvements in accordance with Sections 6602-28 to 6602-30, inclusive. The law provides further for appeal from such an order, either to referee engineers to determine the necessity of the order or to the State Supreme Court by petition in error.

W. M. Olson,* Assoc. M. Am. Soc. C. E. (by letter).—The author makes clear the admirable procedure in Ohio, by which suburban or rural property is assessed for water or sewer mains. Such procedure is impossible in Illinois. Existing laws enable incorporated cities, towns, and villages to construct water and sewer mains and to assess the cost on property benefited. Sanitary districts to provide for sewage disposal through a common outlet may be organized to include incorporated municipalities and unincorporated areas within three miles of included corporations, but no provision is made for the construction, by public means, of water mains or sewer laterals in any unincorporated territory.

The construction of sewers by private capital is possible, but rarely ever practiced because of the difficulty of getting a reasonably large proportion of individual property owners to agree to share the cost. In some cases, real estate operators developing a new area construct some sort of sewer system. Water pipe is sometimes laid by private capital, either by real estate operators, who may construct reasonably adequate systems offering some measure of fire protection; or by "water associations" which lay small pipe to provide domestic water service only to areas adjacent to a public water system.

Unfortunately, the State Department of Public Health is not authorized to exercise adequate supervision over the construction of sewers and water mains. Most municipalities co-operate voluntarily with the Department of Public Health. Experience has demonstrated, however, that private agencies cannot be relied upon to co-operate, and this makes the private construction of sewer and water utilities especially undesirable. To make matters worse, there is no provision for regulations covering zoning, housing, and building construction in unincorporated territory.

As a result of these laws, or this lack of laws, the following conditions may obtain in unincorporated territory: (1) Unscrupulous real estate men and "home builders" may learn that they are free to do practically as they please, and may proceed to do so with a vengeance; (2) entire areas may be built up solidly without sanitary sewerage or proper water supply; and (3) an area may be included in, and taxed by, a sanitary district but may not receive any direct benefit.

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* Res. Engr., Pearse, Greeley & Hansen, Chicago, Ill.

This is just what has happened in many places in Cook County. In 1921, Acacia Park, which was covered largely by truck gardens, was annexed to the Sanitary District of Chicago. About five years later, an ill-advised attempt to incorporate as a tiny village (120 acres) failed. In the absence of proper regulation, it has been a builders' paradise. Houses have sprung up as if by magic. On November 8, 1928, a count of houses within a most densely built-up 80 acres, revealed 370 buildings. Of these, 11 were used for commercial purposes (7 one-story frame, 1 two-story frame, and 3 one-story brick), and 359 were residences (130 one-story frame, 72 one and one-half-story frame, 5 two-story frame, 112 one-story brick, and 40 one and one-half-story brick). Most of the houses are set close together on 30-ft. lots in groups separated by series of vacant lots. There are many good-looking small houses interspersed by a few architectural atrocities. The intensity of use of this area may be indicated by noting that there are 4.6 houses per acre. Zoning ordinances of eight of the better-class suburbs in the county (according to the Chicago Regional Planning Association), specify a maximum population density of 6 families per acre. A house on every lot in Acacia Park will approximate 8 families per acre.

Natural drainage, which is none too good, has been supplemented by farm tile, laid before subdivision, and by makeshift street drains laid since. Existing facilities are entirely inadequate to remove storm run-off promptly. There is a safe domestic water supply distributed privately from the Chicago System, but no fire protection is provided. The fire hazard is enormous.

Practically all the houses have modern plumbing, discharging, supposedly, to individual septic tanks, which, in turn, are connected to the drainage system. Many of the so-called septic tanks are no larger than ordinary kitchen catch-basins, and some houses undoubtedly discharge raw sewage into the road drains.

Extensive building construction has occurred in spite of the fact that reputable banks refuse to loan money for the construction of residences on lots not served by a proper sanitary sewer system, and by a public water supply affording adequate fire protection. Small local banks closely affiliated with building organizations financed the work. The sale of houses was accelerated by the true statement that "You will have no city taxes to pay".

In the spring of 1928, an adjacent area similar to Acacia Park was annexed to the City of Chicago. The Chicago Board of Local Improvements is already making preliminary studies of proposed sewerage and has found that a long and expensive outfall sewer will be required. Acacia Park is particularly unfortunate in not having been included in this annexation, since a comprehensive sewer plan for the entire built-up area probably could have been devised to use a single outlet. As it is, the City of Chicago can make no provision for Acacia Park in its proposed plan, since the law does not permit such construction.

As the result of the lack of sewerage, the usual unpleasant and unhealthful conditions are present. The relatively few basements are flooded after every rain of any consequence. Some enterprising owners have installed electric pumps to discharge the "water" into the alley. In wet weather, heavy trucks break through unpaved roads and crush the shallow drainage lines. Sewage effluent rises to the surface and ponds on adjacent low-lying land. If the breakage of a drainage line causes private septic tanks to overflow, interested persons may repair the break. If not, a complaint may be made to the health officer. Untrapped storm-water inlets emit the odors of decomposing wastes lodged in tile lines laid to no particular grade or alignment and without manholes for cleaning. Even in cool weather, sewage odors may be observed at various places in the community. In hot weather, nuisances abound, and frequent complaints are made to the health officer. In the absence of a sewer system, there is but little for him to do. He cannot indict the whole community. The relative absence of typhoid fever may be explained only by the safe water and milk supplies and by a large measure of good fortune.

With the growth of Acacia Park, conditions will become worse. Private septic tanks, usually installed with the supposition that no cleaning will be necessary, will have to be cleaned to relieve clogging. Nuisances now grievous will become unbearable, and annexation to the city must follow. At that time, the waste of money on temporary sewage disposal will be made clear and will stand out as a convincing indictment of the financial policy which permitted the expenditure of large sums on extensive building operations previous to the construction of necessary sanitary utilities.

The foregoing is intended to show that, in the Chicago Region, the problem of providing necessary sanitary facilities for closely built-up unincorporated areas is acute, and as yet unsolved. New enabling legislation is needed. Procedure similar to that in Ohio should be authorized, or existing sanitary districts should be empowered to construct lateral sewers where necessary and to assess the cost against property benefited. Legislation prohibiting the sale of lots for residence purposes until water and sewer mains have been installed according to plans approved by the Department of Public Health might be justified by modern conditions, but would probably be considered an unwarranted invasion of the rights of private property.

Frank C. Tolles,* M. Am. Soc. C. E. (by letter).—This paper is devoted, first, to an outline of the provisions of the statutes which enable sanitary improvements in suburban Ohio; and, second, to a description of a procedure of assessment. The Enabling Act mentioned by the author is purely local, but in so far as it may be used as a guide for others, it is proper to comment on certain phases of it.

The failure of the 1913 statutes to include provisions for detailed and specific advice to property owners of the terms and of the costs of the undertaking, led to objection on constitutional grounds and resulted in later amendment. The law now requires as a precedent to critical legislation, that there shall be prepared detailed plans, estimates of cost, and tentative assessments.

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* Civ. Engr. with George B. Gascoigne, Cleveland, Ohio.

Individual notice of these is given to each property owner whose address is known. In a project of wide scope this procedure is burdensome. However, and despite the burden of the preparatory work, improvements have been carried on under the Revised Act, although possibly with stricter regard for proved needs. Irrespective of its legal aspect, the statute requirement affords sound protection to the property owner, and it has served to restrict the more speculative developments which, for a time, threatened to bring Ohio sanitary districts into disrepute. From the administrative side, it would be desirable if the preparatory engineering could be lessened. Particularly is this the case in the smaller counties which do not maintain sanitary engineering departments, and in which interest in suburban development is sporadic.

It is noteworthy that the provision of the law for assessing interest charges instead of making these a levy against income, serves to lessen operation costs, and thus favors the direct consumer or user. This feature is helpful in establishing the district as a going concern.

There is still in the law a provision for paying to the county commissioners who authorize the improvement, certain fees which are proportioned on the improvement cost. These fees are small (one-third to one-tenth of 1%, with a fixed maximum), but it would be better if the payments were not so intimately related to the size of the project.

To Mr. Bradbury's statement of the extent of improvements which have been installed under the Ohio law, should be added the fact that, in one or two counties, the auditors' records carry some rather long lists of delinquents in the payments of assessments under the District Act. This fact may reflect a temporary condition in a local real estate market, or it may indicate a too rapid and unwise extension of improvements. This comment is not directed to Franklin County, in which Mr. Bradbury is County Sanitary Engineer.

The Method of Assessing.—The major consideration in the paper concerns a method of apportioning construction costs. Although developed to fit particular statutes and precedents, the method is mathematically, and probably legally, applicable in the general case of spreading costs over a district. To those who have studied this subject, Mr. Bradbury's contribution is welcome as helping to clarify a very "muddled" and unsatisfactory situation. It is, however, open to certain criticisms of practice.

Assessments which proceed from the powers of the Government must be levied in the manner prescribed by enabling laws. The statutes which cover the County Sewer Districts of Ohio recite that, in the case of main works, property which abuts the improvement may be assessed for local service, and that the remaining cost, less any county share, shall be spread as a district assessment on all property in the district and in accordance with special benefits. Other Ohio statutes which relate to municipal improvements provide that special assessments be levied (1) by percentage of tax value; (2) in proportion to benefits; or (3) by the foot front. There is a further provision in the Municipal Code that the sum total of special assessments within a 5-year period shall not exceed one-third the value of the property after the improvements are made. Assessments for other purposes or by other Governmental units are generally by benefits. The mass of legal decisions is found in municipal operations.

In Ohio, the tendency has generally been to apportion assessed costs in proportion to some readily established physical characteristic, including area, frontage, distance from the improvement, or a combination of these. In cases where levies have been made on the basis of benefits, the computation has sometimes been roughly estimated in terms of area or other characteristic and then modified to meet the judgment of the assessor. In other cases, the so-called "point system" has been used to yield a proportionate and relative estimate of benefits. For small improvements on which the assessors are well acquainted with the individual properties this latter method affords reasonably good results.

The Bradbury Formula.—Whereas assessments spread to physical characteristics are readily determinable and comparatively simple of application, those based on benefit must be distributed by exercise of judgment and to meet several factors, which in the aggregate, at least, are rather intangible. Mr. Bradbury's method does not eliminate judgment, but through itemization of elements, focuses judgment on a particular point. In essence, the plan is to make the assessment proportionate to the product of: (1) Ultimate cost less future expenditure; (2) a factor of deferred use; and (3) the square root of a value ratio.

As an expression of individual opinion there is no reason why this method should not be as good as any other reasonable rule. When it is coupled with assessment for local service on a frontage or area basis, the method admits influence by different variables. It, therefore, minimizes the inequities that result from distributing the assessed cost according to any single characteristic. However, any conclusion that assessments computed by Mr. Bradbury's method are more equitable than those determined by physical characteristics alone, is predicated on the assumption that the fundamental data are reasonably accurate. In spreading assessments as described by the author, it is necessary to evaluate certain factors, thus:

(*a*) The total ultimate cost of the improvement when—at some future and unknown date—it is completed.
(*b*) The time sequence—perhaps by 3 or 5-year increments—when different parts of the improvement will be required, and this for a total future period of, say, 20 years.
(*c*) The present value of the land which is to be assessed.

The first of these factors—the eventual cost—is within the range of reasonable estimation, although there should be taken into account the occasional practice, in rural development, of preceding sanitary improvements with pavements. It is conceivable, however, that such estimates would average within an error of 15 per cent.

The time element is rather more difficult to fix; that is to say, for example, that a given sub-district will require trunk outlets or plant facilities in 1945. An error in assigning a sub-area to a 10-year time zone instead of to a 15-year zone might lead to a unit assessment which departs by about 20% from that which might prove more equitable. A rough visualization of this is afforded by comparing the partial assessments in Column (9) of Table 2, for the approximately similar zones *E* 1, *E* 2, and *E* 3, which represent 2-year time differentials.

Assessments are admittedly based on judgment, and it is perhaps not fair to subject the expression of such judgment to more or less mathematical scrutiny, for Mr. Bradbury has undertaken not to point off the decimal, but rather to make determinations rational and comparable. However, the writer has recently made some assessment studies in which the estimate of the time factor was attempted, and he was struck with the discordances of opinion among those whose judgment was pertinent. Faced with the question, "When will this particular area need sewerage", well-informed officials, engineers, and conservative realtors gave answers that differed by ten years, and that further yielded no very clear indication of a preponderance of opinion. In small areas, and with a well marked trend of development, better accord would be expected.

In the matter of setting a value (Mr. Bradbury's third factor), there exists considerable opportunity for variation. Since valuation is itself an expression of judgment, rather than an agreed or barter price, it is subject to all the influences of personal error whether it be from tax duplicate or the result of special appraisal. Also, is the fact that county sewer district improvements are sought by areas which are in process of change from quasi-farm land to sections of suburban development. This betokens a state of flux with a corresponding effect on values, and the resulting difficulties in valuation are great. Moreover, such valuations are those of the moment, during a period when changes are rapid and when there is a wide spread between tax values and holding prices. Neither is it true that, irrespective of the movement of values, their ratios remain unchanged. County sewer districts are generally composed of relatively valuable land which is in immediate and clear need of sanitary improvement, and, further, of acreage which is still under cultivation. The latter is often a large part of the gross area. It is included in the district wisely and (under the present law) in bulk willingly, but the fact remains that its need for service is distinctly of the future. Because of this, there is excellent reason for including valuation or a similar factor as an element of assessment, thus increasing the share to high value lands. However, it is questionable whether, in such cases, values can be estimated to greater precision than with direct evaluation of the resulting benefits.

Table 3 shows derived acreage benefits in a maximum ratio of about 15 to 1 (Areas F5 and A4). This is a greater difference than would result from an apportionment by area, but the writer has spread assessments which have a unit ratio of 12 to 1 without introducing the factor of value.

Conclusion.—Mr. Bradbury has based his plan on the thesis that benefits are measured by the increase in value which results from the improvement. This is true, of course, but it is not at all clear that the enhancement is proportionate to the initial value or to any constant factor thereof.

The writer is impressed with the paper as a workable plan that accounts for pertinent elements in a manner which reflects the matured judgment of an engineer who has labored with the problem. However, the theory is not—

and cannot be—provable. Under conditions in which the variables are determinable within reasonable error, this method should yield satisfactory results, but its use should certainly be coupled with experience and caution in fixing futurities as well as present valuations.

The general subject of special assessments offers a splendid field for engineering analysis and economic thought.* However, a technical passion for precision should not obscure the human elements that are involved.

E. G. BRADBURY,† M. AM. SOC. C. E. (by letter).—The rates charged by the City of Columbus for disposal of sewage from county sewer districts quoted by Mr. Bonney, being based on the number of connections, have the merit of minimizing the necessary accounting between city and county and the work incident to the assessment of the charges against individual property owners. Such rates are not, of course, absolutely equitable but perhaps, on the average, they represent the value of service rendered as correctly as do more complicated methods.

The writer has observed no evidence that unincorporated territory provided with sewer and water improvements is likely to resist annexation to the city. More than one-half the area in which these facilities have been installed by the county has already been annexed to Columbus, and there has been little, if any, opposition by residents or property owners.

The curve for estimating the relation of maximum sewage flow to the average, mentioned by Mr. MacDowell, is also used by the City Engineering Department of Columbus. This fact explains the apparent inconsistency in the statement of Mr. Bonney that the capacities provided by the city are substantially equal to those developed by the basis proposed by the writer, although there is a wide discrepancy between the amounts allowed for infiltration by the two methods. The Morse curve represents the relation of maximum flows to normal quantities in the particular cases observed. The maximum flow is much greater than can be accounted for by fluctuations in the domestic flow and always occurs during or after storms. Therefore, it is evidently due to excessive infiltration or surface water, and is not a function of the domestic flow, as it is made by the use of the curve. It follows that the estimated maximum is unduly affected by the forecast of future population and water consumption. For example, if an area of 100 acres is assumed to have a probable future population of 20 per acre, with a water consumption of 75 gal. per day per capita, and an allowance of 750 gal. per acre per day is made for leakage, the maximum flow as shown by the curve being four times the average, the total quantity per acre is found to be,

$$20 \times 75 = 1\,500 \text{ (domestic)}$$
$$1\,500 + 750 \text{ (leakage)} = 2\,250$$
$$2\,250 \times 4 = 9\,000 \text{ gal. per day}$$

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* An excellent view of current practice in assessing is afforded by bibliographies which are contained in Engineering Societies Library Searches 383, 383a, 2897, and 3555. The Library of Congress also has similar lists. The legal side is covered by the works of Dillon ("Law of Municipal Corporations"), McQuillan (similar title), and, for Ohio, by Page and Jones ("Taxation by Assessment"). Most of the current references bear on assessments for paving.

† County San. Engr., Columbus, Ohio.

After deducting 225% of the domestic flow as a reasonable maximum rate, the actual allowance for infiltration is found to be 5 625 gal. per acre per day.

By the same process, if the population of the district is taken at 30 per acre and the water consumption at 100 gal. per capita per day, and the same figure, 750 gal. per acre per day, used for leakage, it will be found that provision is made for 8 250 gal. per acre per day of infiltration.

If the total normal flow is approximately equal to that on which the curve is based, it will give satisfactory results, but otherwise its use may lead to serious errors. Much of the published information concerning infiltration is misleading because it refers to average leakage rather than to the much higher rate following severe storms. Infiltration is often referred to as being quite constant in quantity, which is far from true. It appears more rational to estimate separately the maximum domestic and infiltration rates, adding for commercial and industrial use when necessary, than to set up an arbitrary relation between average and maximum without consideration of the components.

Mr. MacDowell questions the use of valuation as a factor in apportioning assessments. The writer agrees that this factor cannot be used "with surety that justice is being served," but contends that this is true of any method used, and that justice will be more nearly and more often served if value is taken into consideration than otherwise. The measure of benefit is the enhancement of value of the property. Property located in unfavorable surroundings, and, therefore, low in price, cannot be enhanced in value (in terms of dollars, not in terms of percentage) as much as well-located lots which command prices perhaps several times as great. The use of a factor in direct proportion to value over-emphasizes this condition, but some recognition of relative value is necessary. It is true that a wholly illogical standard of values has been established for platted areas as compared with unplatted land, and owners of the latter may change its status almost over night, thus gaining some advantage over their less astute neighbors. On the other hand, it is to be noted that the owners of platted and developed territory are usually the instigators of the improvement and, in a sense, force it on those whose property is still in acreage and who, for reasons of their own, have not yet seen fit to put it on the market. Among such owners will be found many who are looking forward to later development and others who desire to occupy their land for agricultural or other purposes. There is some justice in placing the greater burden of assessment on those who are asking for the improvement as compared with those who consider it more or less premature in relation to their holdings and accept it only as a necessary evil.

It is desirable that the assessment on the more remote areas, often held in sizable parcels as farm land, be made as low as possible consistent with reasonable equity, and the use of the value factor helps in this direction. There have been several cases in Ohio Courts in which it has been claimed that sewer assessments were confiscatory, and at least two cases in which the Courts have sustained this contention. The writer believes that such a result will often follow if the valuation factor is eliminated, but can hardly occur if value is properly recognized.

The Missouri Sewer District Law, passed in 1927,* provides for the levying of taxes in sewer districts, and for re-adjusting the apportionment from time to time as conditions change. This is an interesting development and, unless too cumbersome in operation, may be an improvement over the Ohio method.

Mr. Olson gives a graphic description of conditions where suburban development is not controlled by proper laws and regulations. Similar conditions (perhaps in a lesser degree) were encountered and cured when the Ohio law came into operation. Real estate operators still sometimes prefer to install sewers and water mains in their developments, but under the Ohio law they are required to do such work according to approved plans, and under county supervision and inspection, the owner paying all expense. In Franklin County such owners are further required to convey such sewers and water mains to the county as soon as completed, for a nominal consideration, for the use and benefit of their property; to place in the County Treasury an amount representing the estimated cost of maintenance for one year; and to authorize the county to levy maintenance assessments against the same property as may be necessary in the future. This procedure was adopted after it was learned by experience that such improvements, if left in the hands of the original owner, are usually entirely neglected after the lots have been sold out, no one being responsible for their maintenance and the county having no funds for the purpose, nor any legal right to operate private sewers or water mains.

Mr. Tolles refers to a provision in the Ohio law which authorizes the payment to County Commissioners of fees based on the cost of the work. This section has been held unconstitutional by the Franklin County Courts, the decision being affirmed by the Circuit Court of Appeals, and is, therefore, no longer effective in this Judicial District, which includes several counties in Central Ohio. How this decision affects the remainder of the State has not been determined.

The various uncertainties and difficulties in establishing time and value factors, referred to by Mr. Tolles, must, of course, exist, but the writer has not found them serious. Assumptions are necessary in many classes of engineering work. The fact that one must estimate future population, water consumption, and infiltration does not justify an arbitrary guess at the necessary capacity of a trunk sewer. The same principle may reasonably be applied to the assessment problem. Intelligent forecasting of the various factors, and a logically developed result therefrom, should ordinarily produce the most nearly correct answer to problems of this type.

In conclusion, the desirability of a method of apportioning assessments to the satisfaction of property owners is self-evident. The Franklin County method has the merit of appealing to the general public as fair and equitable; it is capable of quick explanation in simple language; and it provides something more to rely on than mere personal judgment.

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* Laws, Missouri 1927, p. 439.

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Paper No. 1736

THE PRACTICAL UTILITY OF HIGHWAY TRANSPORT SURVEYS*

By G. F. Schlesinger,† M. Am. Soc. C. E.

With Discussion by Messrs. R. H. Simpson, W. A. Van Duzer, N. W. Dougherty, W. W. Crosby, Charles Randolph Thomas, Jr., and G. F. Schlesinger.

Synopsis

This paper describes how the data obtained in a State-wide traffic and transport survey of the State highways of Ohio were utilized in formulating a plan and budget for future highway improvements. The field information was collected and recorded during a period of one year—December, 1924, to December, 1925. The routes of the State system of highways were classified as to traffic importance, based on the following considerations: (1) Density and character of traffic applied, for convenience of analysis, to five sections of the State; (2) maximum concentration periods; (3) influence of truck and bus traffic; (4) influence of population; (5) traffic service afforded by continuous routes and required by the needs of the individual citizen; and (6) the development and growth of future traffic. The plan of improvement was based on the traffic classification and the existing condition of the highways. A budget of the estimated cost to complete the plan in a 5-year period was prepared.

Procedure for Ohio Survey

A highway transport survey is an accurate and comprehensive collection of traffic data, followed by study and analysis. A number of States, cities, and other political subdivisions have carried on so-called traffic surveys with the object of securing information that would be of value in the solution of their highway problems. The Federal Government, through its Bureau of Public Roads, has taken a leading part in similar investigations. Prior to 1924, it had co-operated with the Highway Departments of California, Connecticut, Maine, and Pennsylvania, and also had made an intensive study of traffic in Cook County, Illinois.

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* Presented at the meeting of the Highway Division, Columbus, Ohio, October 13, 1927.

† Chf. Engr. and Managing Director, National Paving Brick Mfrs. Assoc., Washington, D. C.

A recent project of this kind was a survey of transportation on the State, county, and township road systems of Ohio under a co-operative agreement between the Bureau of Public Roads and the Ohio Department of Highways and Public Works.* Due to the activities of the Bureau, the methods of making such surveys on rural highways have approached standardization. The Ohio survey was begun in December, 1924, and continued for a period of a year, thus covering all seasonal variations. Data were recorded at 1 158 points located so as to obtain the variations in traffic of various routes and sections of routes. Stations were also located on representative secondary county and township roads. At 358 of these points complete data were recorded one day each month during the year period. At the remaining 800 points counts of passenger cars and motor trucks were obtained on three days during the summer months. Data obtained at these stations included a count of passenger cars, motor trucks, motor buses, horse-drawn vehicles, foreign vehicles, and detailed truck and passenger-car statistics. Motor-truck data included the capacity of the truck, State of registration, place of ownership, origin, destination, type of origin and destination, commodity carried, and tire equipment. For alternate periods, at 156 stations, total gross and rear-axle weights were measured by portable scales. Passenger-car data included the State of registration, place of ownership, purpose of trip, origin, destination, and number of passengers.

Each operation consisted of a 10-hour observation period, alternating between 6:00 A. M. to 4:00 P. M. and 10:00 A. M. to 8:00 P. M. Special observers tabulated traffic between 8:00 P. M. and 6:00 A. M. at selected stations, at which, therefore, complete 24-hour observations were obtained. These were made the basis of computation of hourly variations in traffic and of average daily traffic at all stations. Traffic observations for periods of a week were also made at selected stations to determine variations by days. Seasonal changes were computed from the monthly operations at all stations. A carefully planned schedule covered the various days of the week and prevented duplicate recording of traffic.

INFORMATION TO BE OBTAINED

The business of highway departments is to produce transportation service for the users of the highways. A successful head of a business must have full knowledge of his market, that is, reliable information as to the demand for his product, its quantity, and location. A highway transport survey is an effort to supply this information to the engineer executive. Such surveys have been criticized on the grounds that highway authorities cannot make practical use of the data obtained; that the survey is interesting as a speculative thesis, but has no utilitarian value. The principal application of the evidence produced is as the basis of a plan of future highway improvement for a period of years. It is logical that such a plan, evolved to meet the requirements of traffic, should be based on the quantity and character of the traffic that it is designed to serve. This type of development from the Ohio survey will be outlined.

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* The report of this survey was published in the fall of 1927. The highway traffic studies upon which it is based, were conducted under the joint supervision of Thomas H. MacDonald, Chief of the Bureau of Public Roads, U. S. Department of Agriculture, and the writer, then Director, and L. A. Boulay, former Director of the Ohio State Department of Highways and Public Works. J. Gordon McKay, Chief of the Division of Highway Economics, Bureau of Public Roads, directed the work of the survey and preparation of the report, assisted by Messrs. O. M. Elvehjem, E. T. Stein, L. E. Peabody and B. P. Root, all of the Division of Highway Economics, and Messrs. Harry J. Kirk, Ohio State Highway Engineer, and Harry E. Neal, Ohio Traffic Engineer.

The density and character of traffic that existed in 1925 were determined in the field as described. The expected future increase was estimated, and the routes of the State system were classified as to their traffic importance. On this classification was based the plan of improvement for the following 5-year period.

The term, "density of traffic," means the number of motor vehicles passing any given point on a highway in a unit of time. The accuracy of determining it is influenced by the distance between the survey stations. Exactness of method would require a density record for each point on the highway system at which traffic varies. The cost involved, in proportion to the relatively small gain in accuracy, does not justify such a close location of observation points. The density computed for each station on the Ohio Highway System is applied to the short sections of highway that are reasonably adjacent, on which the traffic varies but little.

Variable Highway Conditions

The State highway system of Ohio consists of 11 000 miles of the total of 84 884 miles of rural highways. It is estimated that in 1925 there was a motor-vehicle movement of approximately 3 746 000 000 vehicle-miles on all rural highways and 2 160 000 000 vehicle-miles on the State system. In other words, 13% of the mileage carried 58% of the total motor-vehicle traffic. On the basis of vehicle-miles per mile of highway, the traffic on the State system is more than nine times as intense as that on the secondary rural highways.

The daily volume of traffic on different parts of the State highway system varies widely, as is shown graphically in Fig. 1. Of the 11 000 miles of the State highway system, 131 miles, or 1.2% of the total mileage, carried at the rate of 2 500 or more motor vehicles per day in 1925; 858 miles (7.8% of the system), carried 1 500 or more vehicles per day; 3 239 miles (approximately 30% of the total), carried 600 or more vehicles per day; and 7 761 miles (70.6%), carried less than 600 vehicles per day, of which 4 180 miles carried less than 200 vehicles per day. These relations are shown in Fig. 2.

A basic traffic map of the State was prepared. This is somewhat similar to Fig. 1 and shows the details of distribution of passenger-car and motor-truck traffic, the estimated 1930 traffic, the traffic classification, and the density

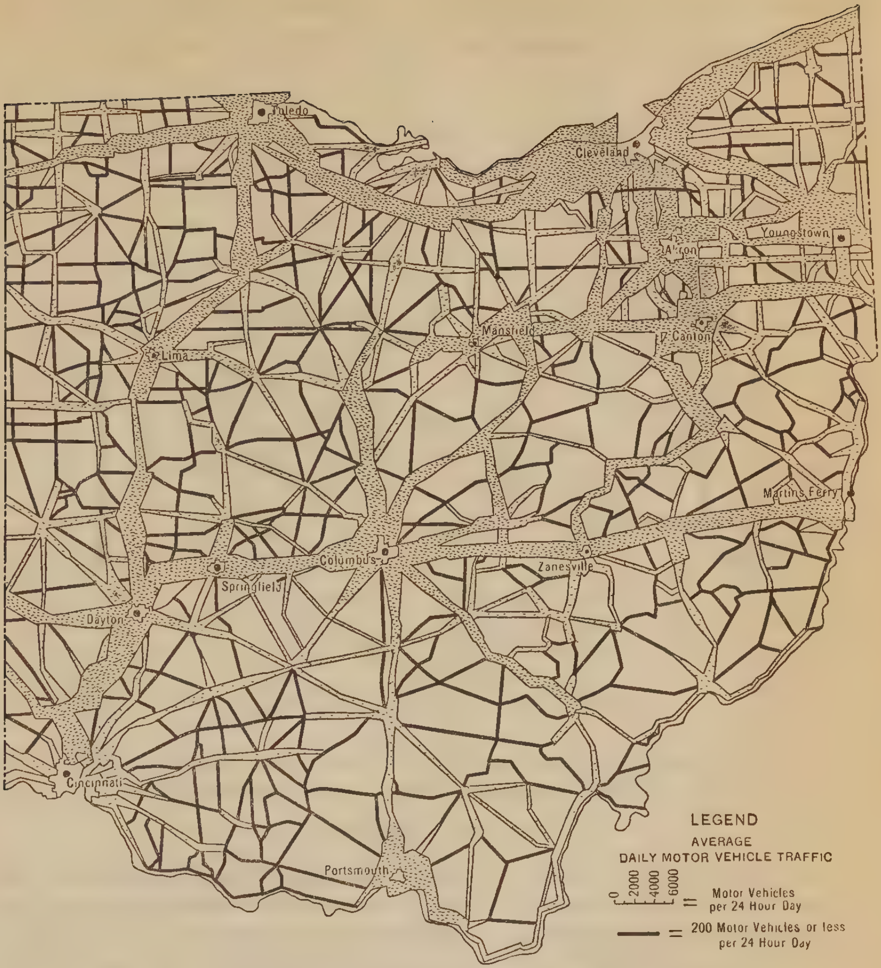
Toledo
Cleveland
Youngstown
Akron
Mansfield
Canton
Lima
Martins Ferry
Columbus
Zanesville
Springfield
Dayton
Cincinnati
Portsmouth
LEGEND
AVERAGE
DAILY MOTOR VEHICLE TRAFFIC
0
2000
4000
6000
= Motor Vehicles per 24 Hour Day
= 200 Motor Vehicles or less per 24 Hour Day

FIG. 1.—MOTOR VEHICLE TRAFFIC ON OHIO HIGHWAY SYSTEM.

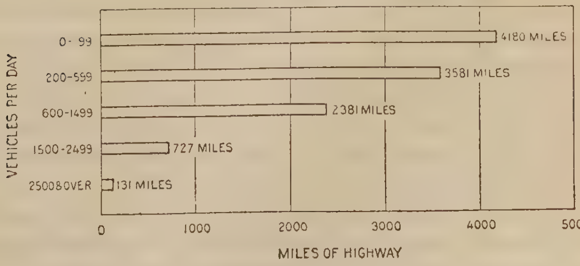

FIG. 2.—MILEAGE OF STATE HIGHWAYS BY TRAFFIC DENSITY CLASSES.

and trend of population by townships. In its preparation, allowance was made for routes under construction and in poor condition at the time of the survey.

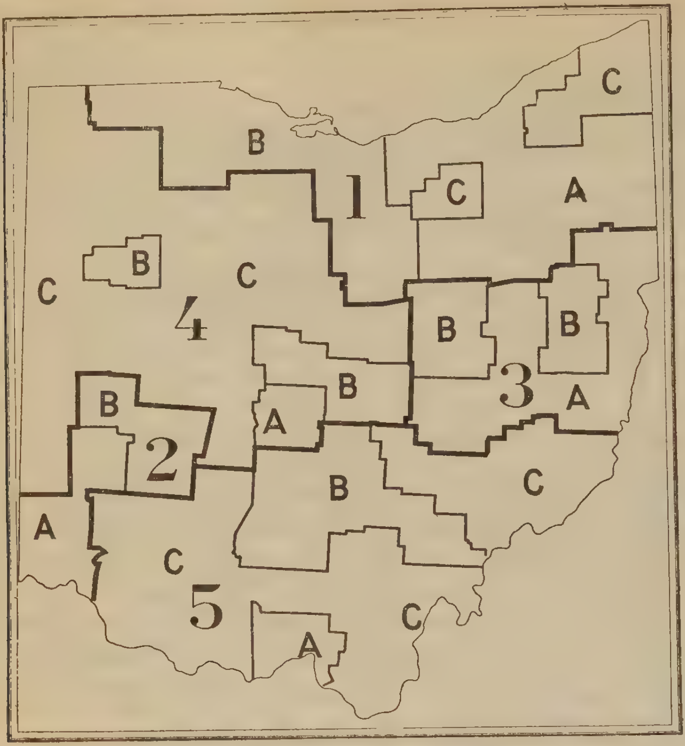

FIG. 3.—OHIO TRAFFIC SECTIONS AND THEIR SUB-DIVISIONS.

DENSITY OF TRAFFIC

For convenience of analysis the State has been arbitrarily divided along county lines into five traffic sections showing distinctive characteristics as to traffic, population, topography, and industrial development, each of these sections being subdivided in the order of its traffic importance, into two or more divisions, as shown in Fig. 3, which if compared with Fig. 1 will show the location of the traffic sections with reference to traffic density, population, and industry. The relative traffic importance of the sections is shown in Table 1, wherein the mileage of highways in the five sections is classified according to the density of traffic in 1925.

Within the northeastern and the southwestern sections (Nos. 1 and 5) the routes carrying 600 or more vehicles per day include almost one-half the mileage. In the northwestern section, also, similar routes serve to connect the more important cities, as well as to form a series of short sections radiating from these centers. More than two-thirds of the State highway mileage in this section, however, has a daily traffic of less than 600 vehicles.

In the southern area, sections of highways carrying from 600 to 1 500 vehicles per day comprise only 15.0% of the State highway mileage in the area; and this relatively small percentage serves the more important cities.

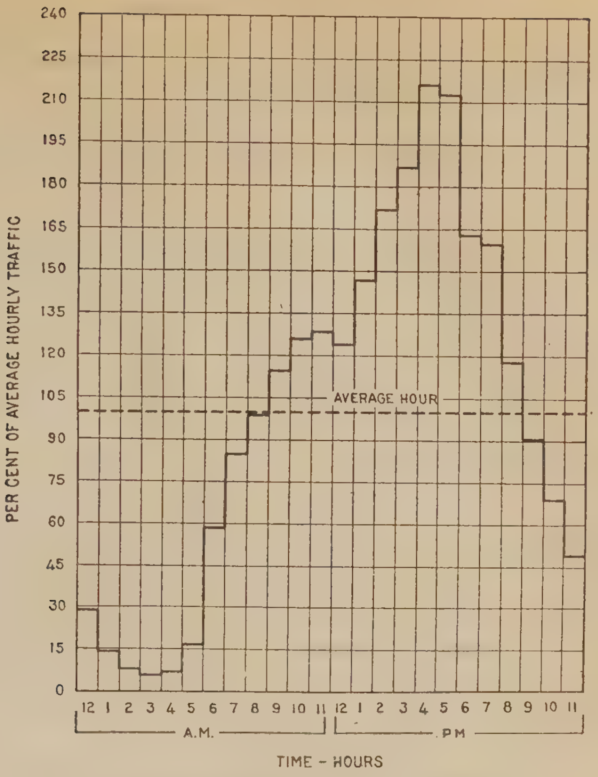

FIG. 4.—HOURLY VARIATION OF TRAFFIC EXPRESSED AS PERCENTAGE OF TRAFFIC DURING THE AVERAGE HOUR.

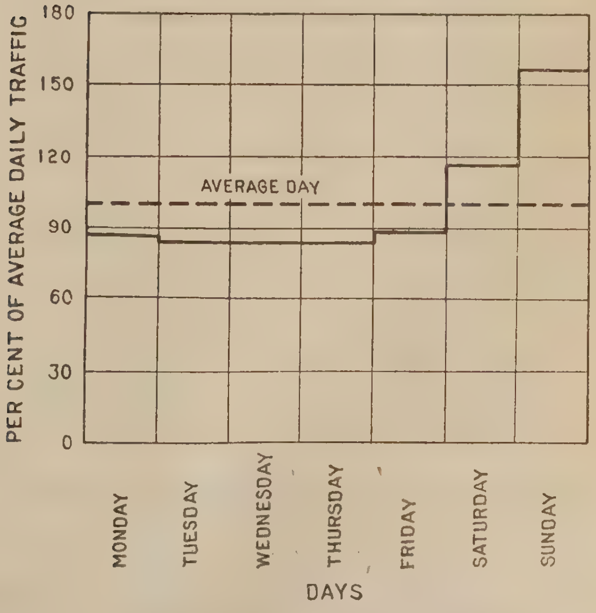

FIG. 5.—DAILY VARIATION OF TRAFFIC EXPRESSED AS PERCENTAGE OF TRAFFIC DURING THE AVERAGE DAY.

TABLE 1.—Mileage of Ohio State Highways by Traffic Classes and Traffic Sections (Fig. 3).

Section.		All State Highways.		Daily Traffic, More Than 1 500 Vehicles.		Daily Traffic, 600 to 1 500 Vehicles.		Daily Traffic, Less Than 600 Vehicles.	
Number.	Location.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.
1.....	Northeastern......	2 821	25.6	454	52.9	836	35.1	1 536	19.8
2.....	Southwestern.....	736	6.7	133	15.5	202	8.5	401	5.2
3.....	East-central.......	1 285	11.7	86	10.0	265	11.1	934	12.0
4.....	Northwestern.....	3 402	30.9	162	18.9	679	28.5	2 556	32.9
5.....	Southern..........	2 756	25.1	23	2.7	399	16.8	2 334	30.1
......	State total........	11 000	100.0	858	100.0	2 381	100.0	7 761	100.0

The traffic density for an average 24-hour day by no means represents the peak concentration of traffic. Traffic varies with the season, month, day of the week, and hour of the day. As shown by the survey (Figs. 4, 5, and 6) the normal maximum traffic on a typical road in Ohio would occur on a Sunday in August at 4:00 p. m. By combining the data on these graphs this is found to be approximately 480% of the annual daily average. In the vicinity of the larger cities the maximum concentration of traffic necessitates consideration of multiple-lane surfaces and parallel routes.

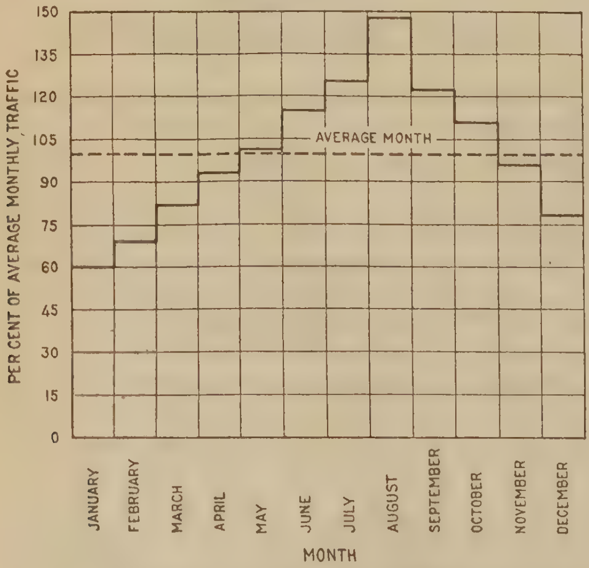

Fig. 6.—Monthly Variation of Traffic Expressed as Percentage of Traffic During the Average Month.

Influence of Truck Traffic

The density and character of motor-truck traffic, as distinguished from passenger-car traffic, must be known in the planning of a system as well as in the design of pavements. It was found to vary greatly on different routes and in various parts of the State. While it averages only 9.5% of the total on the State system, its importance is due to the greater load concentrations, the use of solid tires (15% of all trucks), and a lack of refinement in springs and shock-absorbing parts.

The relative importance of the five traffic sections from the standpoint of truck traffic is shown in Table 2 and Fig. 7. The influence of large cities on density of truck traffic is shown in the two most important trucking areas, the northeastern and southwestern sections, which have five of the seven cities between 30 000 and 100 000 population.

TABLE 2.—TRUCK AND POPULATION DATA IN OHIO.

Section.	Average truck traffic density.	Percentage of total area in the State.	Truck registration per square mile (1924).	Cities of More Than 10 000 Population by Population Classes.*				
				Total.		10 000 to 30 000.	30 000 to 100 000.	More than 100 000.
				Number.	Percentage.			
Northeastern....	77	24	7.9	22	44	15	3	4
Southwestern....	75	6	10.9	7	14	3	2	2
East-central.....	53	12	2.5	9	18	9	....	....
Northwestern....	36	30	2.3	7	14	5	1	1
Southern.........	36	28	1.3	5	10	4	1	....
Total........	51	100	3.9	50	100	36	7	7

*U. S. Census of 1920.

It was found that the rated capacity of trucks was a reliable criterion of the weight of the total gross load, and that the proportion of the total gross load on the rear axle increases with an increase in capacity of the truck.

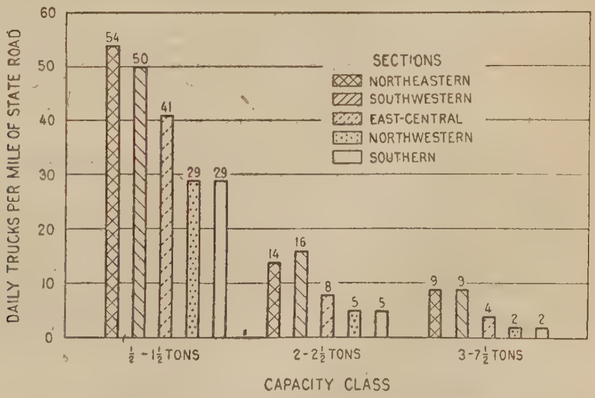

FIG. 7.—DISTRIBUTION OF MOTOR TRUCKS BY CAPACITY CLASSES IN THE FIVE TRAFFIC SECTIONS OF OHIO.

The highway engineer, therefore, must provide for increased load concentrations on routes where large capacity truck traffic is a factor. Only about 2.1% of the trucks with a rated capacity greater than 5 tons were operating on the highways (see Fig. 8). This was evidently due to Ohio's legal maximum gross load limitation of 20 000 lb. (10 tons). It is also in line with the general tendency to prefer medium-sized units as the highway transportation vehicle. It was found that, in general, the heavier gross loads occurred on routes carrying a high density of truck traffic, although there were exceptions to this general rule accounted for largely by special movements, such as the hauling of gravel, sand, and stone for construction work.

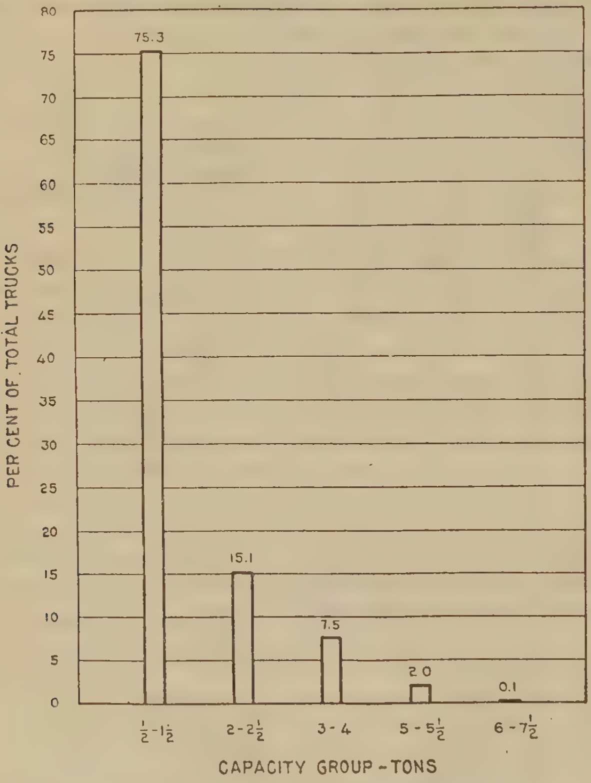

FIG. 8.—DISTRIBUTION OF MOTOR TRUCKS BY CAPACITY GROUPS.

Motor-bus traffic was noticeable. It is relatively the most rapidly increasing type of highway transportation, being largely limited to the State highway system and amounting to 1.5% of the passenger-car traffic during 1925. This traffic must be considered in highway planning, necessitating a high type surface of greater width than would be required by other types of traffic.

HIGHWAY TRAFFIC AND POPULATION

The volume of highway traffic in an area is largely a function of the population of the area. A study of traffic and population shows, that the concentration of traffic is greatest in the areas tributary to the large cities of the State.

Analysis of the trip-mileage of motor vehicles in Ohio shows that 60.4% of the cars observed, travel less than 30 miles per trip, and approximately 70.0%, less than 50 miles; also, that 71.6% of the trucks travel less than 30 miles, and only 14.0%, more than 50 miles per trip. Highway traffic is, therefore, primarily a method of local transportation.

TABLE 3.—ANALYSIS OF MOTOR TRAFFIC IN OHIO.

Section.	Persons per square mile, 1920.	Motor vehicles per square mile, 1925.	DAILY VEHICLE MILES ON STATE HIGHWAY SYSTEM.				Persons* per motor vehicle.
			Per square mile.	Per person, 1920.	Per motor vehicle, 1925.	Per mile of State highway.	
I.—Northeastern:							
A	434.5	101.8	292.4	0.67	2.87	1 000	5.13
B	149.5	38.3	204.3	1.37	5.33	684	4.28
C	67.8	18.1	157.6	2.32	8.70	549	3.92
Total	264.1	63.5	236.5	0.89	3.72	804	4.87
II.—Southwestern:							
A	601.4	126.5	270.2	0.45	2.14	944	5.08
B	130.4	30.7	175.6	1.35	5.72	601	4.53
Total	373.7	80.2	224.4	0.60	2.80	776	4.98
III.—East-Central:							
A	131.2	26.5	165.8	1.26	6.26	614	5.25
B	46.6	10.4	66.9	1.43	6.34	281	4.50
Total	101.2	20.8	130.7	1.29	6.29	506	5.11
IV.—Northwestern:							
A	549.2	145.3	259.2	0.47	1.78	1 144	4.25
B	99.1	24.6	199.3	2.01	8.12	723	4.22
C	62.6	15.3	101.0	1.61	6.61	363	4.14
Total	87.5	21.9	119.7	1.37	5.48	434	4.18
V.—Southern:							
A	100.9	24.4	157.3	1.56	6.44	801	4.67
B	71.2	15.2	100.0	1.40	6.56	446	4.75
C	55.0	10.4	74.0	1.34	7.09	291	5.29
Total	61.2	12.3	84.4	1.38	6.87	345	5.07
State total	141.4	32.5	145.3	1.03	4.46	538	4.79

* Based on estimated population, 1925.

A comparison of the factors producing traffic in the five traffic sections is shown in Table 3. The three divisions having the greatest density of population also have the greatest volume of traffic per square mile, but the traffic on the State system is not in the same ratio to population or motor-vehicle registration as in the other sections. This variation is due principally to the following causes:

(1) In the populous sections a larger proportion of the traffic is on city streets instead of on State highways.

(2) In the populous sections the roads other than those on the State system carry a greater proportion of the traffic.

(3) Traffic originating in the populous sections uses highways in other sections to a greater extent.

A county or area should not be required to bear a major portion of the financing of the improvement of a through route for the benefit of traffic that is not of local origin. The distribution of population affects the planning of highway improvements in several important particulars. Densely populated areas require pavements of a width and type that will serve adequately and expeditiously the large volume of traffic, including trucks and buses, between important centers of population. On such routes, separation of grades at railroad crossings, reduction of heavy grades, revision of poor alignment, elimination of curves around parks in small villages, construction of multiple-lane and parted pavements, and the avoidance of traffic deterrents in general are economically justifiable. An adequate plan of highway facilities in such areas requires the provision of "by-pass" routes around cities, which not only are of benefit to traffic on the main routes, but also decrease traffic on the city streets by drivers unacquainted with local conditions.

Although a rigid application of the evidence does not justify the inclusion of as large a mileage of through highways in the less populated sections, the principle of traffic service makes a connected system of routes in such areas a proper function of the State. As shown in Table 3, the vehicle-miles on the State system per person and per motor vehicle in such areas are greater. It would follow, therefore, that the State routes are of greater service to each individual resident of these sections. It is true, however, that highways of width in excess of a normal two-lane road are rarely required and low-type surfaces of the traffic-bound type will give ample service for years in the future.

FORECAST OF FUTURE TRAFFIC

Any sound plan for highway improvement should take into consideration the traffic of the future in so far as it can be predicted with reasonable accuracy. Ohio had no historical series of traffic counts previous to 1925. From a study of such records of varying length and accuracy, from Maine, Maryland, Massachusetts, Michigan, and Wisconsin, it is evident that in these States, highway traffic has increased directly with the increase in motor-vehicle registration. This assumption was adopted for the Ohio forecast. The registration for a 10-year period until 1935 was estimated by computing future population, using the Bureau of Census method and by extending the persons-per-car curve, the former, of course, having an upward and the latter, a downward trend. By combining the estimated population and the estimated number of persons per car the predicted registration for each year is obtained. The results are shown in Table 4.

In forecasting traffic in 1930 at each traffic station, the procedure was similar to that for predicting motor-vehicle registration in each county, the county rate being applied to each traffic station in the county. For a number of reasons such refinement was not considered justifiable in the forecast for 1935 which was computed on the basis of the State increase in registration rather than on that of the increase in each county.

The methods described will not reflect certain changes in suburban and industrial growth that may occur and influence traffic on short sections of highways. There are other conditions that cannot be anticipated and taken into account. The Ohio Department of Highways is continuing the traffic survey with a skeleton organization, that is, gathering data at certain key stations of the original survey. This work was inaugurated in December, 1926, two years after the original survey was begun.

TABLE 4.—COMPARISON AND PREDICTION OF POPULATION AND NUMBER OF MOTOR VEHICLES IN OHIO.

Year.	Registration (Thousands).		Population* (thousands).	Persons per Car.	
	Actual.	Estimated.		Actual.	Estimated.
1913	86	86	5 095	59.24	59.24
1914	123	127	5 197	42.60	40.92
1915	181	179	5 299	29.28	29.60
1916	252	245	5 402	21.44	22.05
1917	347	324	5 504	15.86	16.99
1918	413	416	5 606	13.57	13.48
1919	511	521	5 708	11.17	10.96
1920	621	637	5 810	9.36	9.12
1921	721	763	5 913	8.20	7.75
1922	859	897	6 015	7.00	6.71
1923	1 069	1 038	6 117	5.72	5.89
1924	1 242	1 181	6 219	5.01	5.27
1925	1 346†	1 329	6 321	4.70	4.76
1926	1 480†	1 475	6 424	......	4.36
1927	......	1 621	6 526	......	4.03
1928	......	1 763	6 628	......	3.76
1929	......	1 902	6 730	......	3.54
1930	......	2 035	6 833	......	3.36
1935	......	2 607	7 344	......	2.82

* Population as of July 1, of each year.

† Data not available when forecast was made. Estimate differs by 1.3% from actual value in 1925, and by 0.3% in 1926.

TRAFFIC CLASSIFICATION OF STATE HIGHWAYS

Having determined the present and future traffic on the State system of highways the next step in formulating a plan of highway improvement was to classify the routes as to their traffic importance. A highway should be designed and constructed so that it will economically serve present and future traffic. The principle of stage, or progressive, construction contemplates the improvement of the roadway with a surfacing that will be increased in strength as the traffic grows. Such a policy has been resorted to—and justifiably so—by highway authorities when funds were not available at the outset for the construction of a durable type of pavement. However, in order that the highways included in a proposed program may be neither over-designed nor under-designed, they should be divided into traffic groups.

The State highways of Ohio are classified in three groups designated as major, medium, and minor, according to their average daily traffic. Highways classed as "major" include, in addition to the roads carrying 1 500 or more vehicles in 1925, those sections which carried less than 1 500 in 1925, but which are expected to carry 1 500 or more vehicles in 1930; similarly, in 1930, in addition to the sections actually carrying at that time 1 500 or more vehicles there are included as major traffic highways those which are expected to carry that number in 1935.

A similar method is utilized in classifying the highways of the medium traffic group, which includes sections actually carrying between 600 and 1 500 vehicles per day in 1925 and expected to carry that number in 1930. Minor traffic highways, on the other hand, are those which are expected to carry less than 600 daily vehicles in 1925 and 1930.

The traffic limits selected are in line with general highway engineering practice and were confirmed by a study of maintenance costs on various types of surfacing in Ohio. A traffic density of 600 vehicles per day is the limit at which traffic-bound gravel or stone roads can be economically maintained. For greater intensities this type should be strengthened with bituminous treatment or additional courses. A traffic density greater than 1 500 vehicles per day calls for a standard pavement design—the flexible or rigid types for the lower densities, and usually rigid pavements only for roads carrying large volumes of traffic with the accompanying frequency of heavy loads.

The mileage of major and medium classification in 1930 which is not now improved with surfaces superior to gravel affords a reliable index to the need for new improvements during the next few years. The quantities shown in Table 5 include all highways now having such surfaces, regardless of condition or present width. Included in this mileage is a considerable length in poor condition which must be reconstructed to provide adequate highway service, and a large mileage of narrow surfaces which must be widened in order to serve present and expected future traffic.

TABLE 5.—Mileage of Surfaces Superior to Gravel in 1925 and 1930.

Section.	Mileage of major and medium classification, 1930.	Mileage of major and medium classification improved with surfaces superior to gravel, 1925.	Mileage of major and medium classification not improved with surfaces superior to gravel, 1925.
Northeastern	1 985	1 160	825
Southwestern	469	282	187
East-central	547	323	224
Northwestern	1 370	620	750
Southern	850	303	547
Total	5 221	2 688	2 533

In the east-central, northwestern, and southern sections more than one-half the total State highway mileage is expected to remain in the minor traffic class in 1935.

The Ohio Improvement Plan

Having determined a traffic classification of the highways on the State system considering traffic needs of the future, the plan of improvement was formulated based on the existing condition of the highways. The mileage of proposed new construction, reconstruction, and widening in the five traffic sections is shown in Table 6. In the improvement plan for a 5-year period from January 1, 1927, to December 31, 1931 (Table 7), new construction is defined as construction on present unimproved sections of highways and sections where the surface cannot be salvaged because of its condition, location, or alignment. The selection of surfaces for this new construction will be based on traffic and physical conditions. Reconstruction is defined as the rebuilding of worn-out surfaces with the same or a superior surface type. On the other hand, widening is the extension of present surfaces to a minimum of 18 ft. and to a greater width where required. In general, surfaces 16 to 18 ft. in width on minor traffic routes are not included in the widening program. If the condition of the surface requires reconstruction such roads have been included in the reconstruction program. The distinction between widening and reconstruction is not clear in many cases, but for the larger part of the reconstruction mileage the condition of the surfaces is such as to require rebuilding. Where the surfaces are narrow, reconstruction also will include widening of the surface.

TABLE 6.—PROPOSED IMPROVEMENT PROGRAM.

Section.	Total State highway mileage.	Total Improvement Program.		New Construction.				Reconstruction.		Widening.	
				Surfaces Superior to Gravel.		Traffic Bound.					
		Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.	Miles.	Percentage of total miles.
Northeastern......	2 821	1 511	53.6	295	10.5	167	5.9	496	17.6	553	19.6
Southwestern......	736	269	36.6	53	7.2	...	...	88	12.0	128	17.4
East-central.......	1 285	574	44.7	122	9.5	124	9.6	132	10.3	196	15.3
Northwestern	3 402	1 360	40.0	252	7.4	332	9.8	306	9.0	470	13.8
Southern...........	2 756	807	29.3	285	10.3	77	2.8	198	7.2	247	9.0
Total..........	11 000	4 521	41.1	1 007	9.1	700	6.4	1 220	11.1	1 594	14.5

The cost of each of the three classes of improvement was obtained by estimating the changes for grading, minor structures, and the approximate surface type required for the sub-grade conditions and estimated future traffic. Exclusive of bridges and the separation of railroad grade crossings, this will amount to approximately $100 000 000.

The urgent need for widening of present surfaces is indicated by the fact that there were on the State system, in 1925, approximately 4 800 miles of surfaces superior to gravel and less than 18 ft. in width, approximately 1 400 miles being from 10 to 15 ft. wide. On light traffic routes, where the present width is between 16 and 18 ft. and the surface is in good condition, widening can well be deferred to a later period and, therefore, this mileage is not included in the program. The plan also includes a limited mileage of present 18-ft. pavements, which require additional width to serve existing and expected traffic.

TABLE 7.—PROPOSED OHIO FIVE-YEAR NEW CONSTRUCTION, RECONSTRUCTION, AND WIDENING PROGRAM AND ESTIMATED IMPROVEMENT COST.

Class of improvement.	Miles.	Percentage.	Estimated cost.	Percentage.
New construction	1 707	37.8	$41 122 000	41.2
Surfaces superior to gravel	1 007	....	$34 443 000	....
Traffic-bound	700	....	6 679 000	....
Reconstruction	1 220	27.0	35 188 000	35.2
Widening	1 594	35.2	23 644 000	23.6
Total	4 521	100.0	$99 954 000	100.0

The 5-year new construction, reconstruction, and widening program involves 41.1% of the total State highway mileage. The remaining 58.9% (6.479 miles) consists of surfaces in satisfactory condition and adequate for expected traffic during the next five years.

TABLE 8.—ESTIMATED BUDGET, OHIO STATE HIGHWAY SYSTEM, JANUARY 1, 1927, TO DECEMBER 31, 1931.

Item.	Estimated total cost.	Funds required by State Highway Department.
New construction, reconstruction, and widening of highway surfaces	$100 000 000	$100 000 000
Bridge construction	8 000 000	8 000 000
Railway grade-crossing elimination	16 000 000*	8 000 000
Maintenance and repair	53 000 000	53 000 000
Administration	3 000 000	3 000 000
Contingency fund	5 000 000	5 000 000
Total	$185 000 000	$177 000 000

* To be shared equally by the State and railroad companies involved.

The reconstruction and widening program is largest in the heavy traffic sections which contain the greatest amount of major traffic mileage. The proposed program for new construction superior to gravel is relatively large in the southern, or lowest traffic, section because of the present small mileage of such types, and it is limited mostly to a few important traffic routes.

An estimate of budget requirements for the 5-year period is shown in Table 8. The estimate given in Table 7 did not include such items as the cost of major bridges, railroad grade-crossing eliminations, routine maintenance, contingencies, and administration. The elimination of grade crossings presents a serious problem when it is considered that there are more than 1 000 railroad crossings at grade, outside of cities, on the State system.

CONCLUSION

An endeavor has been made in this paper to describe how the results of a transport survey have been utilized, in a practical manner, by establishing a plan of State highway improvement for a period of years, and preparing a budget required by this plan. Other information made available by a transport survey is useful in many ways, but might be classified as being outside the scope of practicality. For example, in the Ohio survey the composition of passenger-car traffic as to state of registration, touring or non-touring, business or non-business, city or farm ownership, was obtained. The fact that the survey evidence indicated that 87.6% of the passenger-car traffic on the State system was of city ownership had a decided influence on the type of measures adopted by the Legislature of 1926-27 for financing improvements on the State highway system. Other information obtained included such data as: Average mileage per trip for passenger cars and trucks; average number of passengers per car; trucks for hire and privately operated; principal commodities hauled by commercial trucks; classification of commercial and private trucks according to capacity; and a comparison of motor truck and railroad tonnage between selected Ohio cities.

The results of the survey and the plan based thereon were presented to the 1927 General Assembly and were effective in obtaining sizable appropriations for State highway improvements, although not quite as large as called for by the budget (Table 8). The main benefit to be expected from the establishment of the plan is that the Ohio Highway Department now (1927) has a definite aim toward which it can direct its future efforts. The fact that the Federal Highway Bureau was a party to its formulation, and that it has been endorsed by the non-political organizations of the State interested in highway progress and the highway industry, would seem to insure its inviolability as the future road program for Ohio.

DISCUSSION

R. H. Simpson,* M. Am. Soc. C. E.—The author has shown very clearly that a knowledge of how the highways are used is just as essential to the man who directs the activities and policies of a highway system, as a knowledge of the market for a commodity is to a business executive. Engineers have devoted much time and study to the details of pavement construction, and yet it is only recently that they have made an effort to find out the use and abuse to which roads and streets are subjected.

In the design of a bridge, careful consideration is given to the loads to be sustained. Is it not important, therefore, to know the character and density of traffic to be carried in order to decide the type and to construct a pavement properly? A lack of such information in past years has not only led to the selection of types entirely inadequate to handle the volume of traffic carried, but in some cases to the selection and construction of types, the cost of which cannot be justified by traffic conditions. Engineers are beginning to recognize, however, that a knowledge of present, and an intelligent forecast of future, traffic is essential, if they are to design properly and to construct roads and pavements economically.

Since a large percentage of the traffic on the main highways of the State originates in the cities, it will be of interest to know something of the movement of traffic in the City of Columbus, Ohio. The Ohio survey shows that approximately one-eighth of the rural roads carries more than one-half of the total road traffic. What percentage of city traffic is carried by the main thoroughfares within that city? What is the relation of traffic to the paved mileage in a city; what is its intensity; its fluctuations, and its character? It may not be necessary to know the seasonal variation, but it is desirable to know the direction and maximum intensity of traffic in order to plan intelligently the development of a city. The general direction of traffic will indicate clearly the need of widening present thoroughfares, and will show where additional arteries of traffic may be required.

To obtain a complete and accurate record of the flow of vehicular traffic over the streets of a city presents a problem of some difficulty. It is manifestly impractical to secure information as to ownership, origin, and destination of traffic on a thoroughfare carrying 1 000 vehicles per hour, and such information is hardly necessary in solving problems arising from city traffic. This special problem is rather one of density and calls for information as to direction, total volume, maximum intensity, and something as to the character of traffic.

Traffic counts at best give only an indication of traffic movement within reasonable variation. A count to-day will not agree with one to-morrow or yesterday. The density of traffic at a particular point will vary with the seasons and with weather conditions. It will vary with the day of the week and the hours of the day. Notwithstanding the inexactness of the data, a survey of traffic is very important for the engineer who must plan a system of highways as well as those who must solve the traffic problems in cities.

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A few isolated traffic counts were made in Columbus about 1924, but the first systematic survey was undertaken in 1925. The data obtained from this effort, while not very complete, convinced the speaker of the need of continuing the work during the following year. In 1926 a more complete and thorough survey was attempted. The work was under the direction of R. C. Chaney, M. Am. Soc. C. E., Engineer for the City Planning Commission. It was begun on June 14 and continued daily, except Sunday, until September 28. The standard forms prepared for this work enabled the observer to record all traffic movement at a station, so that the movements of any 15-min. interval would be known. Most of the stations were at street intersections, so that the record would show the movement straight through on each street and the right and left turns. The data noted included a count of passenger cars, light and heavy trucks, and horse-drawn vehicles. Data were obtained at 210 stations selected to cover the entire city. The observation period was from 6:00 A. M. to 10:00 P. M. Some Sunday counts were taken at a few stations. At a number of selected stations a complete 24-hour count was taken, and, at a few, the movement was observed for seven consecutive days. Many of the stations were handled by a single observer, but on heavily traveled thoroughfares, two observers worked together.

While the survey covered a 16-hour day, a 13-hour day was adopted for the purpose of this study, from 6:00 A. M. to 7:00 P. M., as a standard interval best suited to give a dependable average traffic. This period covers the business day, both commercial and commuting, and the traffic during this interval is fairly uniform throughout the year. The traffic for this interval is about 70% of the 24-hour traffic.

There are in Columbus about 540 miles of dedicated streets. Of this total, 400 miles are improved. The remainder of the mileage is not improved, so that practically all the traffic movement of the city is carried by the 400 miles of improved streets. The roadways vary in width from a minimum of 18 ft. to a maximum of 80 ft., the average being about 30 ft.

Of this paved mileage, 48.12 miles, or 12.03%, may be classed as main thoroughfares, arteries that lead from the center of the business district in all directions to the suburban sections; 38.25 miles, or 9.56%, may be classed as secondary thoroughfares; and 313.62 miles, or 78.41%, as minor streets. The average daily movement (13-hour interval) on these roadways is as follows:

Main thoroughfares.......	7 000 vehicles, or	336 875 car-miles
Secondary thoroughfares..	3 434 vehicles, or	131 370 car-miles
Minor streets............	410 vehicles, or	125 450 car-miles
Total daily movement....................		593 695 car-miles

This equals a movement of about 9½ miles per day for each registered car in the city.

It is interesting to note that the main and secondary thoroughfares, representing 21.59% of the paved mileage, carry 81% of the traffic movement. It will be noted, also, that the main thoroughfares, representing 12.03% of the paved mileage, carry 56.7% of the traffic movement. This fact is rather striking in that it agrees closely with figures presented by Mr. Schlesinger on the relation of traffic on the State routes as compared with the total rural traffic movement. In other words, the traffic movement on the main thoroughfares of the city bears about the same relation to the total traffic in the city streets as the traffic on the State routes bears to the total rural traffic.

The character of the traffic on city streets is of some importance. The Columbus survey shows that an average of 16% of the vehicular movement is motor trucks, and that approximately 25% of these trucks were classified as heavy, so that about 4% of the traffic consists of heavy trucking units. The movement of horse-drawn vehicles is negligible because less than 1% of the traffic is of this class.

Figs. 9 and 10 show the variation of traffic in the city compared with that of the State highways, the data on the latter being from the Ohio survey as presented in Fig. 6 by Mr. Schlesinger. Fig. 9 shows the hourly variation expressed as the percentage of traffic during the average hour. It will be noted that the intensity of highway traffic increases rather uniformly from a minimum at 4:00 A. M. to a maximum at 4:00 P. M. The traffic intensity in the city, however, while rising rapidly in the morning, is fairly uniform from 8:00 A. M. to 4:00 P. M. The maximum intensity occurs about 6:00 P. M., followed by a slump with another peak in the early evening.

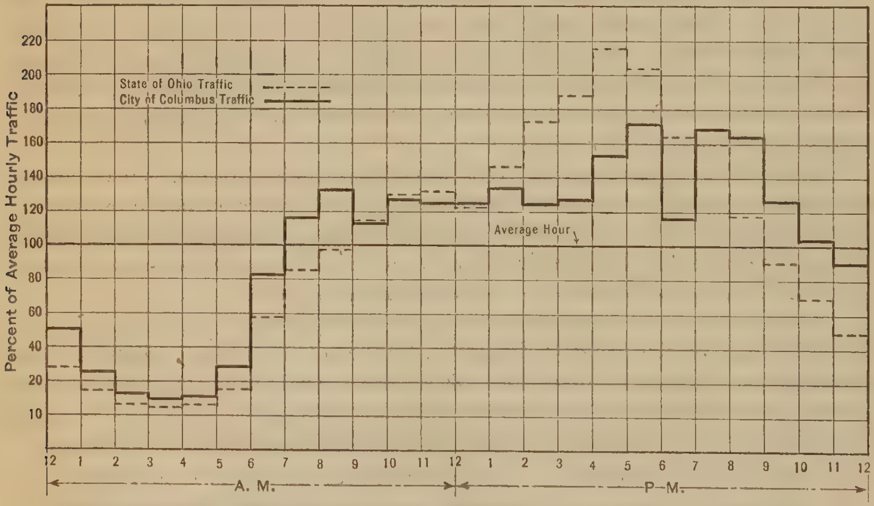

FIG. 9.—HOURLY VARIATION OF TRAFFIC EXPRESSED AS PERCENTAGE OF TRAFFIC DURING THE AVERAGE HOUR.

Fig. 10 shows the daily variation of traffic expressed as percentage of traffic during the average day. The traffic intensity on the highways reaches a maximum on Sunday, while in the city it is a minimum on that day. On the highways, the Saturday and Sunday traffic is more than the average and, in the city, the traffic intensity exceeds the average on Monday and Saturday.

Studies in Columbus have not been carried to a point where a complete thoroughfare plan can be outlined for the needs of the city, but in a number of locations it has furnished evidence to show the need of wider roadways and new thoroughfares. The traffic survey has been an aid in determining the proper location and the proper width of arteries extending into newly developed territory and in planning the development of suburban areas. It has also furnished evidence to aid in the selection of locations for the installation of automatic traffic signals.

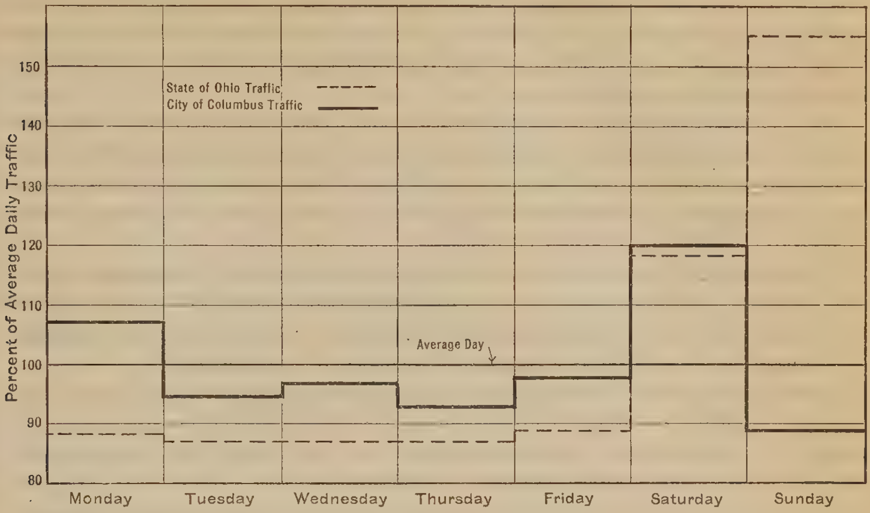

FIG. 10.—DAILY VARIATION OF TRAFFIC EXPRESSED AS PERCENTAGE OF TRAFFIC DURING THE AVERAGE DAY.

The traffic on the streets should also be studied in an effort to determine its effect on pavements. What is the relation of traffic to the wear and repair of pavements? A few years ago this question was of little moment, but the use of self-propelled vehicles has reached such proportions as to show its effect on pavement surfaces. To-day, many hard-surface pavements which carry a large volume of traffic are showing abrasive wear, especially where the traffic is concentrated in narrow lanes and has no opportunity to spread over a wide area. A test of a concrete pavement by the U. S. Bureau of Public Roads, at Arlington, Va., showed an abrasion under concentrated traffic of nearly 0.01 in. by the passage of a vehicle 55 000 times over the surface. At the same ratio 1 000 000 vehicles would have resulted in a wear of 0.17 in. Some observations by the speaker on a pavement in Columbus, carrying 5 000 000 vehicles yearly, shows that rubber-tired traffic will abrade hard surfaces, and there is evidence to show that the abrasion is approximately proportional to the volume of traffic. When it is realized that many pavements carry a traffic in excess of 2 000 000 vehicles yearly, it is clear why many of them are showing signs of distress, and why such vast sums are required for maintenance. Is it not important, therefore, to have a knowledge of the traffic conditions in order not only to plan the future development of rural and urban communities intelgently, but to design and construct pavements properly? The time is past when pavements should be constructed without reference to the use to which they are to be placed. The engineer should know within reasonable limits the density and character of the traffic to be carried. The only way to secure this information is by a traffic and transport survey.

W. A. Van Duzer,* M. Am. Soc. C. E.—As Mr. Schlesinger states, the principal application of evidence produced from highway transport surveys is as bases of plans for future highway improvement. Furthermore, the value of a transport survey is cumulative. A great deal of the evidence may be of little or no utility at the time it is gathered, but may become valuable in the future, in combination with later data indicating trends important of consideration in connection with making or modifying plans for improvement, or in connection with traffic control.

The principal information that is secured from the highway transport survey is as to distribution and annual daily average of traffic flow, wheel loads, and peak loads; and the greatest value of this information is in its use with trend curves. Past records and present information with future trends furnish a conception of future conditions.

The author gives a very clear description of the Ohio Transport Survey and its uses. Summarizing, and at the same time generalizing, the speaker would say that such a survey furnishes information pertinent to the economic development and profitable and safe utilization of the highways.

Priority of Improvement.—Information as to distribution of traffic flow makes it possible to classify the highways of a system for priority of improvement. Mr. Schlesinger indicates this and he also states that motor highway traffic is of recent and incomplete development.

In twenty years, highway traffic has changed from horse-drawn to motor-driven and in volume has probably increased fivefold. Since 1908 the necessity for greater strength and durability has increased the cost per mile of pavement construction approximately five times, while the demand for more mileage has increased in even greater proportion.

At present (1927), the pressing task is to provide a durable, hard surface on a vast mileage of trunk highway systems, and the highway transport survey may be considered as furnishing data for use in solving the problem of priority of improvement.

In general, the total volume of traffic, as, for instance, when expressed in total vehicle-miles, is a measure of the value of road service. In order to determine the merit or efficiency of any road system, it is necessary that its value to the user—which may be considered as the saving in operating over improved, rather than unimproved, roads—be capable of comparison with the cost of furnishing improved roads.

As an illustration, it is estimated that total travel on the Pennsylvania State Highway System in the next four years will average about 10 000 000 000 vehicle-miles per year; that there is a 25% saving in operating over improved road, equivalent to 2½ cents per mile for passenger-car travel and 9 cents per mile for motor trucks and buses. Assuming 10% motor-truck and bus traffic, the annual saving is about $225 000 000 for passenger cars and $90 000 000 for motor trucks and motor buses, or a total of $315 000 000 per year for all traffic. It is very important that the capital charges plus upkeep of the State highway investment show a profit to the public. Admittedly, estimated unit savings are open to question; but the figures appear to be reasonable, and until the road costs approach such total as would be equivalent to savings computed by using such lower savings figures as might be advanced, the question as to accuracy of the estimate is not critical. The more definite determination of savings figures, however, may prove important in the future and it is through traffic surveys and studies that the improvements in accuracy can be obtained.

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Location.—The improvement of any route, or any section of a route, raises questions which can only be disposed of satisfactorily by comparison of costs. If, for example, there are two alternative locations, one of which will cost $30 000 per mile for grading and the other $20 000, and if the larger expenditures result in a shortening of distance or a reduction of maximum grade, the number of vehicles using the road will determine which of the two is the preferable and economic location.

Selection of Type.—The information as to distribution and annual daily average (Table 1) is valuable in the solution of the problems of highway economics in determining the type of improvement.

It is an engineering precept that there should be an estimate of load to be carried, by any structure, throughout its economic life. In the past years traffic has been changing at such rapid rate—while adequate standards of construction were being developed—that forecasting has been virtually impossible; but now that the rate of increase has moderated, a curve of future traffic can be drawn with some logical basis. There are now modern types of pavement which have been carrying traffic a sufficient length of time to warrant estimating the length of their economic life. The relation between volume and weight of traffic, and costs of construction and maintenance of pavement, has become sufficiently stabilized so that it can be observed and studied to verify or correct the assumptions on which the present huge expenditures on highways are based and on which in the future greater expenditures are to be planned.

Design of Pavement Section.—Information as to weight of heavy wheel loads and their frequency makes it possible to provide a pavement section of sufficient strength to withstand—without destructive cracking—the loads to which it will be subjected. Conversely, information as to what wheel loads are being carried without serious damage, and what wheel loads are excessive for various types of pavement, will furnish evidence to verify or correct present ideas concerning requirements of cross-section and materials.

Information as to volume and nature of peak loads to be carried and capacity of multiple-lane or parallel roads, under traffic characteristic of locality, is also essential for planning, intelligently, the width of road surfaces. There may not be general agreement as to what should be considered the maximum carrying capacity of two, three, or four-lane roads, under different conditions. It is within the last few years, and only in a comparatively limited mileage, that carrying capacity of two-lane roads has been exceeded, and even now the composition of traffic is still changing. There is, as yet, a wide variation in proportions of motor-truck and bus traffic to be considered in the present as well as estimated in the future. These considerations, however, emphasize rather than detract from the importance of traffic survey data in connection with the planning of improvements.

Time Factor.—The principal features of the highway transport survey have been considered to be those which most directly affect the road itself. This has been the point of view necessarily taken during the early stages of development of the road system. With the road system approaching completion, the point of view changes to embrace operation as well as construction. In the future, highway administration may be expected to concern itself more and more with operation and traffic control.

Time is an ever-increasingly important factor. Pavements might withstand the full weight of traffic and carry the full volume of it and yet be unsatisfactory if travel entailed loss of time. It is necessary to provide highway facilities for fast as well as slow travel, as in the case of railroads. In the near future, widening of roads with special lanes for slow travel, or segregation of slow travel to parallel roads, for the purpose of reducing congestion, is likely to occasion large expenditures and, in such cases, adequate information will be required.

With the understanding that road design is to permit occasional over-loads, in the same way that storm sewers are designed to be occasionally overtaxed, the peak load of traffic, expressed in hourly rate, rather than annual daily average, should be the measure of highway capacity. The planning of multiple lanes and parallel roads will be influenced accordingly. Width of right of way as well as width of pavement is involved.

In many cases, especially on trunk roads entering cities where it may not be possible to provide either extra width or parallel roads, unsatisfactory conditions may be remedied by eliminating or reducing to a minimum those features which retard the flow of traffic and constrain the volume-carrying capacity of the road. Such features, concerning which a traffic survey may furnish very valuable information, are railroad crossings, cross-road or side-road intersections, sharp curves, and steep grades.

TABLE 9.—1926 STATE HIGHWAY ACCIDENT HAZARD BY DAY OF WEEK.

Day.	Percentage of accidents.	Percentage of traffic.	Factor.	Relative hazard.
Sunday	18	24.5	73	100
Monday	15	12.1	124	170
Tuesday	13	11.1	117	160
Wednesday	11	11.8	93	127
Thursday	12	12.1	99	136
Friday	12	12.8	94	129
Saturday	19	15.6	122	167
Total	100	100.0	....	....

Safety.—The purpose of the highways is their safe and profitable use. Safety requires, essentially, jurisdiction over drivers and minimum specifications, particularly as to brakes and lights. The growing accident loss is to be checked and the proper use of the highways is to be encouraged. This feature of highway traffic study must receive a great deal more consideration in the future and traffic-flow data must be combined with accident-occurrence data to furnish an accurate conception of the problem which is prerequisite to remedy.

Table 9 furnishes an example. It shows that in 1926, of accidents reported as occurring on the State highways of Pennsylvania, the percentage charged to Sunday was greater than that for any other day of the week, but when daily average of traffic is taken into account, it is found that Sunday is the day of lowest relative hazard. In connection with the analysis of accident data, it is obvious that the number of accidents is significant and important, but it is also obvious that comparison is valuable; and in this case a comprehensive comparison must include the indication of accident rate in terms of total volume of traffic. Public welfare requires traffic control, and traffic control necessitates traffic information.

N. W. Dougherty,* M. Am. Soc. C. E. (by letter).—Highway transport surveys, as described by the author, have a special value in connection with the construction of highway bridges that are to be financed by toll charges. In 1927, the State Legislature of Tennessee authorized the construction of sixteen toll bridges. The legislation comprises three acts, each providing for a series of bridges to be considered as a unit and to be financed by the issuance of bonds, which, in turn, are to be retired by the income from tolls.

The bridges in the first group are needed and the tolls will easily finance their construction and maintenance. The bridges of the second group are in isolated areas and on secondary highways and it is doubtful whether any of them will prove to be paying investments. As a whole, this series will be difficult to finance from the tolls collected. The acts authorizing these two groups also fixed the charges that will be made, thus fixing the income which the Highway Department may reasonably expect from the structures. Had studies of income been made before the legislation was enacted, several of the structures would not have been authorized because it would be cheaper to operate ferries without charge. The bridges of the third series will serve a real traffic need and at least two of them will finance themselves. It is probable that the two profitable bridges will finance the third.

All the bridges in each group were classed together, and were to be paid for out of the tolls, the group to be made free as soon as the bonds on the whole series were retired.

Estimate of Traffic.—The State Highway Department of Tennessee has been conducting studies for a period of years in order to determine the relation between the average flow and the seasonal flow of traffic as well as the volume of traffic that may be reasonably expected over the proposed bridges. The data show that the traffic in August is approximately 33% more than the average for the year. The total 24-hour traffic is 133% of the daylight flow. This was determined by taking night counts. A 12-hour count taken in August, therefore, approximates the average 24-hour flow throughout the year.

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In 1926 a general count was made throughout the State, and this is used as a basis for predicting future travel.

In Tennessee, as in other States, the increase in traffic flow has been approximately proportional to that of motor-vehicle registration. This has been used as a basis of foretelling the volume of future traffic. Predictions made about 1923 have agreed fairly well with the actual registration for 1926 and 1927.

A more accurate value may be determined if the many sources are estimated separately. A study of vehicle license numbers from the several sections of the State has shown that the preponderance of travel is from the county in which the road is located or from the adjoining counties. The study was divided, therefore, into: (*a*) Local travel; (*b*) travel from counties of large registration; (*c*) travel from other counties of the State; and (*d*) travel from other States. No effort is made to separate the truck flow from the travel of automobiles, on the assumption that the same law applies for the trucks as for the passenger car.

Local Travel.—To estimate local travel, a large number of stations were selected at which the flow of vehicles was obstructed as little as possible. It is well known that detours, bad stretches of road, construction under way, and other obstructions, limit the free flow of vehicles. From preceding counts an estimate was made of the flow of local vehicles at the stations. From records of motor-vehicle registration, an estimate was made of tributary motor vehicles by assuming that they would pass the station in proportion to their remoteness considering only the counties in tributary areas. In other words, of the total cars registered in the various counties, a certain percentage will contribute potentially to the daily traffic density at a fixed observation station. The writer has estimated this percentage as some ratio of the relative distances or remoteness from the observation station.

A typical analysis is shown in Table 10 for the station at Spring Hill, Tenn., near the Williamson-Maury County line. The percentages in Column (2) varied with the distance from Spring Hill and from a knowledge of the topography in the area.

The local travel at Spring Hill in 1926 was 573 vehicles per 12-hour day, which was a ratio of travel to weighted registration of 0.091. If two main thoroughfares are parallel across a county, certain vehicles will be tributary to one highway and others will flow over the other highway. About twenty stations were studied in a similar way, and it was found that the weighted motor-vehicle registration was ten times the local travel at the stations.

It is assumed, therefore, that the travel at other unobstructed stations will be approximately one-tenth the tributary motor-vehicle registration (see Table 11). An estimate was then made of the motor-vehicle registration tributary to all the proposed bridge sites. This was done by projecting the curve of registration from 1920, through 1927, to 1940. Many factors cause the registration to change, but all of them can hardly be estimated in advance. The trend of the preceding seven years was probably the best information at hand and, therefore, was used in the study. The population was estimated from the Census reports and the registration was taken from the records of the Department of Finance and Taxation through 1927.

TABLE 10.—ESTIMATED NUMBER OF REGISTERED VEHICLES THAT CONTRIBUTE TO TRAFFIC DENSITY AT SPRING HILL STATION.

County. (1)	Percentage of total cars registered that contribute to traffic. (2)	Cars registered in county. (3)	Weighted values. (4)
Williamson	80	2 908	2 326
Maury	60	3 164	1 898
Giles	30	2 655	797
Lawrence	30	2 315	695
Marshall	20	1 917	283
Hickman	10	1 115	112
Lewis	10	644	64
Total	....	......	6 175

The weighted value taken is that obtained by assuming that the percentage of the cars tributary to the station varied with the distance from the source to the station. Some of the percentages are somewhat arbitrary and were determined from a personal knowledge of the topography and characteristics of the sites.

TABLE 11.—ESTIMATED RATIO BETWEEN NUMBER OF CARS PASSING A STATION AND TOTAL CARS REGISTERED IN THE DISTRICT SERVED.

Route.	Location.	Cars registered.	Number of cars counted.	Ratio.
1	Blountville	6 900	692	0.100
24	Overtown County Line	2 150	210	0.097
1	West of Crossville	2 130	230	0.108
6	Spring Hill	6 275	573	0.091
3	Ripley	6 220	685	0.110
28	Dunlop	1 340	149	0.104
16	East of Shelbyville	6 125	613	0.099
64	One mile south of Madisonville	1 200	60	0.050
34	Seven miles west of Morristown	3 400	350	0.103
1	Church Hill	4 300	500	0.125
10	Ten miles south of Murfreesboro	4 160	500	0.120
52	Macon County	2 400	280	0.115
1	McEwen	3 400	350	0.103
6	Near Franklin	5 500	600	0.110
76	Seven miles south of Clarksville	2 300	175	0.075
3	South of Union City	4 500	360	0.080
42	Modina	5 000	360	0.072
1	Huntingdon	7 000	610	0.087
Totals, or average.		74 360	7 289	0.097

A typical analysis is given in Table 12 for Route 15 at the bridge over the Tennessee River between Waynesboro and Selmer, Tenn. (Bridge No. 3, Fig. 11).

Weighted registrations were obtained in a similar manner, using the actual registrations for each county each year (see Table 13).

The local unobstructed travel, therefore, is estimated at 340 vehicles in 1930, 385 in 1935, and 400 in 1940.

Large Cities.—The four large counties contain 48.5% of the motor vehicles registered in the State. It is desirable, therefore, to study the effect of motor-vehicle concentration in the large cities. Fortunately, information was available for this study. During the 1926 count, automobile license numbers were listed at more than forty stations. From the records in the Bureau of Registration, the location of ownership was determined and a tabulation was made for Davidson, Knox, Hamilton, and Shelby Counties, which include the Cities of Nashville, Knoxville, Chattanooga, and Memphis, respectively. Fig. 11 (*a*) shows by contour the number of cars per day originating in Davison County as they traveled throughout the State of Tennessee in 1926. The composite map (Fig. 11 (*b*)) shows the effect of nearly 50% of the automobiles of the State on the travel at the several bridge sites.

TABLE 12.—TYPICAL ANALYSIS OF POPULATION TRAFFIC AT THE BRIDGE AT SAVANNAH, HARDIN COUNTY, TENN.

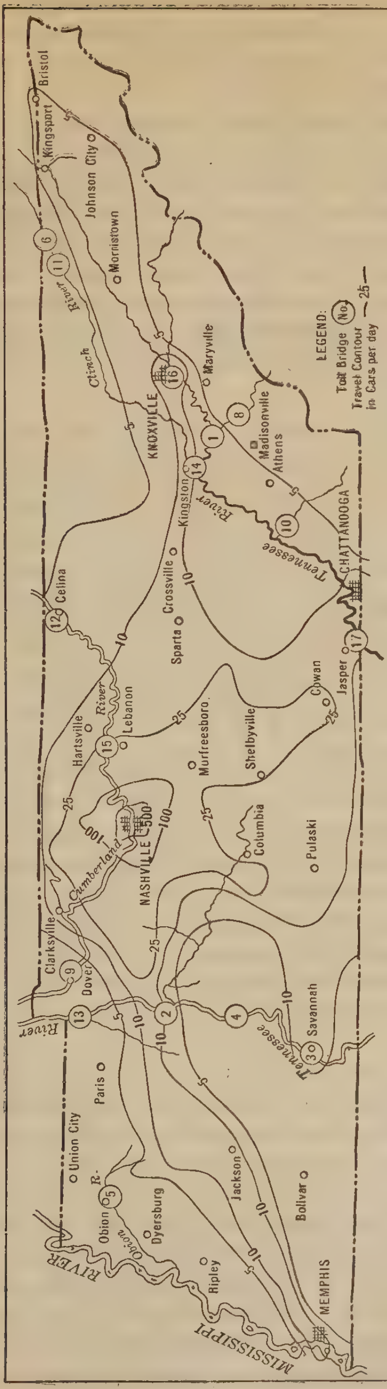
County.	Percentage of total registration effective.	Population.		Estimated Population That Contributes to Traffic at the Savannah Bridge.	
		1910.	1920.	1910.	1920.
Hardin.........	90	17 521	17 291	15 769	15 562
McNairy.......	70	16 356	18 350	11 449	12 845
Wayne	50	12 062	12 877	6 031	6 439
Lawrence......	10	17 569	23 593	1 757	2 359
Chester........	10	9 090	9 669	909	967
Hardeman.....	10	23 011	22 278	2 301	2 228
Total.........	....	......	......	38 216	40 400
Increase in 10 years	....	......	......	......	2 184

By projecting the registration for the four large counties forward to 1930 and 1940, an estimate may be made of the probable travel over the several bridges from this source.

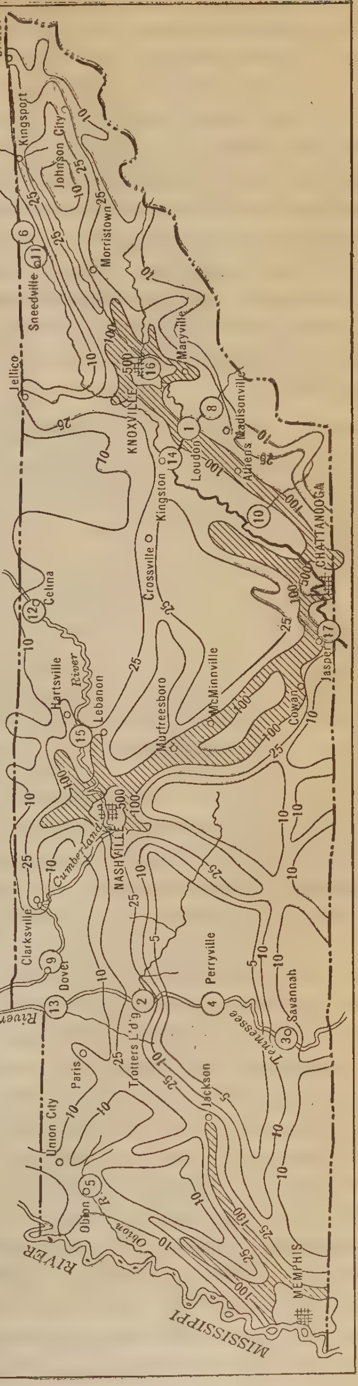
TABLE 13.—WEIGHTED REGISTRATIONS FOR STUDY OF SAVANNAH BRIDGE.

Year.	1910.	1920.	1921.	1922.	1923.	1924.	1925.	1926.	1927.	1930.	1935.	1940.
Population................	38 216	40 400	40 618	40 836	41 054	41 272	41 490	41 708	41 928	42 580	43 670	44 760
Number of cars registered.	......	756	837	892	930	1 465	2 030	2 638	2 828	3 400	3 850	4 000
Persons per vehicle.......	......	52.5	48.5	45.5	44.0	28.0	20.3	15.9	14.8	12.5	11.6	11.2

Bridge No. 1 at Loudon, for example, is between Knoxville and Chattanooga, on the primary system (Fig. 11) and a large flow may be expected. Bridge No. 6, at Kyles Ford, is not on the primary system; it lies in a remote section of the State, and, consequently, serves very few vehicles from the large counties.


Bristol
Kingsport
Johnson City
Morristown
Clinch River
KNOXVILLE
Maryville
Madisonville
Athens
Kingston
Tennessee River
CHATTANOOGA
Celina
Crossville
Sparta
Hartsville
Lebanon
Cumberland River
NASHVILLE
Murfreesboro
Shelbyville
Cowan
Jasper
Columbia
Pulaski
Clarksville
Dover
Paris
Union City
Obion R.
Dyersburg
Ripley
Jackson
Bolivar
Savannah
MEMPHIS
MISSISSIPPI RIVER
LEGEND:
Toll Bridge (No.)
Travel Contour in Cars per day 25
1
2
3
4
5
6
8
9
10
11
12
13
14
15
16
17
5
10
25
100
500

(*a*) DAILY TRAFFIC OF CARS REGISTERED IN DAVISON COUNTY


Bristol
Kingsport
Johnson City
Morristown
Sneedville
Jellico
Maryville
KNOXVILLE
Madisonville
Athens
Loudon
Kingston
Crossville
CHATTANOOGA
Celina
Hartsville
River
Lebanon
McMinnville
Murfreesboro
Cowan
Jasper
Cumberland
NASHVILLE
Clarksville
Dover
River
Perryville
Trotters L'd'g
Tennessee
Savannah
Paris
Union City
Obion R.
Jackson
MEMPHIS
RIVER
MISSISSIPPI
1
2
3
4
5
6
8
9
10
11
12
13
14
15
16
17
5
10
25
100
500

(*b*) ASSEMBLY OF TOTAL DAILY TRAFFIC OF CARS REGISTERED IN DAVISON, HAMILTON, KNOX AND SHELBY COUNTIES

FIG. 11.—TRAFFIC MAP OF TENNESSEE FOR 1926.

The travel from other counties not included in the local and large cities has been assumed arbitrarily as 50% of that from the counties of large registration. The registration for the four counties is projected into the future, so as to give an estimate for 1930, 1935, and 1940 (Table 13).

Fig. 12, which is typical of the studies made, shows a steady decrease in rate of registration. This is explained by the fact that, at present, there is 1 motor vehicle to each 5.4 persons in these counties. It is probable that the density will not increase to much more than 1 motor vehicle to 4.5 persons. Some States have a denser registration, but they also have a greater wealth per capita than Tennessee. The average wealth of the United States is double that of Tennessee. In Fig. 12, the registration is the weighted value, derived by estimating the percentage of total cars registered throughout the State that may be expected to influence traffic density at the observation station under discussion.

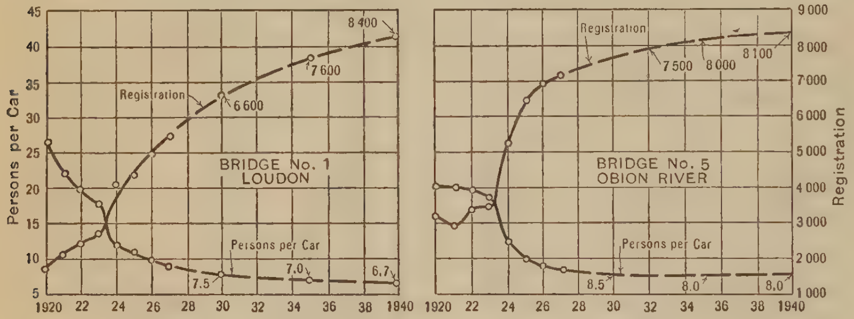


FIG. 12.—TYPICAL CURVES OF WEIGHTED REGISTRATION.

Foreign Travel.—On the main thoroughfares crossing the State there has been a marked increase in outside State travel since about 1926. This increase will continue as the highways are improved, but it is difficult, if not impossible, to estimate the future rate. The amounts entered in Table 14 are based on the present flow, modified by a knowledge of previous increase and by the location of the several bridge sites. For example, at Savannah (Bridge No. 3, Fig. 11), it is assumed that the travel will increase greatly in the near future because of the attraction of Shiloh National Park, and because of the improvement of State highways in this area.

Charges at Bridges.—The law provided a schedule of tolls at each of the structures in Series No. 1 as listed in Table 15. From traffic information taken at each of the bridges, a weighted charge per motor vehicle is computed. The estimate of future traffic is based on motor-vehicle registration, and, consequently, the income is based on motor vehicles alone. At each of the sites a few horse-drawn vehicles still use the highways. There is a varying distribution of light and heavy trucks, depending on the location. At each bridge an estimate of income has been made from actual travel, according to the schedule of the tolls listed in Table 15, and from this estimate the average charge per motor vehicle is made. The Tennessee cars are kept separate from the foreign cars because the percentage of trucks from outside

TABLE 14.—ESTIMATED DAILY TRAVEL ON BRIDGES OF THE FIRST SERIES.

Territory.	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 8.
1930							
Local	660	415	340	205	750	134	370
Large counties	200	45	12	6	15	6	30
Other counties	100	20	6	3	8	3	15
Total, in State	960	480	358	214	773	143	415
Foreign	400	200	75	20	100	20	35
Total	1 360	680	433	234	873	163	450
Probable ratio of actual to theoretical traffic flow	80	55	50	35	90	30	60
1935							
Local	760	450	385	235	800	148	420
Large counties	220	50	14	8	20	8	35
Other counties	110	25	7	4	10	4	20
Total, in State	1 090	525	406	247	830	160	475
Foreign	600	300	150	40	250	30	60
Total	1 690	825	556	287	1 080	190	535
Probable ratio of actual to theoretical traffic flow	90	60	60	40		40	70
1940							
Local	840	480	400	250	810	153	480
Large counties	230	60	15	10	25	10	40
Other counties	115	30	8	5	13	5	20
Total, in State	1 185	570	423	265	848	168	540
Foreign	750	400	250	50	400	40	80
Total	1 935	970	673	315	1 248	208	620
Probable ratio of actual to theoretical traffic flow	100	70	75	50	100	50	75

TABLE 15.—TOLL CHARGES SPECIFIED ON BRIDGES IN SERIES NO. 1.

Item No.	Description.	Bridges Nos. 2, 3, and 4.	Bridges Nos. 1, 5, 6, 7, and 8.
1	Automobile and driver	$0.50	$0.25
2	Person, each	0.05	0.05
3	Motor truck, or motor bus (1 ton capacity or less) and driver	0.50	0.25
4	Motor truck or motor bus (more than 1-ton capacity) and driver	1.00	0.50
5	Automobile trailer	0.25	0.15
6	Motor-truck trailer or motor-bus trailer	0.50	0.25
7	Motorcycle and driver	0.25	0.15
8	One-horse vehicle, with draft animal and driver	0.15	0.10
9	Two-horse vehicle and two draft animals and driver	0.25	0.15
10	Extra draft animals or horses or mules, each	0.05	0.05
11	Cattle, sheep, hogs, or other domestic animals, other than horses or mules on foot, each	0.02	0.02
12	Circus animals, other than those specifically mentioned in Items 8, 9, 10, and 11	0.25	0.25

the State is very small. The average charge for Tennessee motor vehicles is obtained by dividing the total receipts for the day by the total motor vehicles passing the station. The charge for foreign automobiles is obtained by adding to the amount received for the automobiles, one and one-half times 5 cents, making the charge for Bridges Nos. 2, 3, and 4, equal to 57.5 cents, and for Bridges Nos. 1, 5, 6, and 8, a charge of 32.5 cents. A sample computation is given for the Loudon Bridge (No. 1, Fig. 11), as follows:

Automobiles	640 × $0.25 =	$160.00
Passengers	1300 × 0.05 =	65.00
Light trucks	32 × 0.25 =	8.00
Heavy trucks	35 × 0.50 =	17.50
Trailers	1 × 0.25 =	0.25
Horse-drawn	1 × 0.15 =	0.15
Horse-drawn	5 × 0.10 =	0.50
640 + 32 + 35 = 707 vehicles =		$251.40
707 × 35.5 + =		251.40
$0.25 + 5 × $0.015 =		$0.325

The average number of passengers per car in Tennessee, as shown by previous counts, is 2.5 persons per car. Data taken at the ferries indicate a similar number of passengers. An automobile, therefore, will pay 25 cents for driver and car and 5 cents additional for each passenger.

To obtain the daily estimated tolls for 1930, 1935, and 1940, Tennessee travel is multiplied by the average charge for each station, and the foreign travel is multiplied by 32.5 cents, or 57.5 cents, according to the schedule of tolls fixed by law. The weighted charges for each bridge in Group 1 are listed in Table 16.

TABLE 16.—WEIGHTED TOLL CHARGES FOR BRIDGES IN GROUP 1.

Description.	Charge per vehicle.						
Bridge No.	1	2	3	4	5	6	8
Local	$0.355	$0.650	$0.575	$0.610	$0.340	$0.330	$0.325
Foreign	$0.325	$0.575	$0.575	$0.575	$0.325	$0.325	$0.325

Expectancy.—In making an estimate of the travel at the several bridges, free flow was assumed from all the tributary area. This assumption is far from the truth for many of the bridges proposed. Some of the locations are in areas where highways will have to be constructed. The estimates made from the curves, therefore, are modified as nearly as may be to suit the station in question.

Traffic was counted in August, 1927, at all the stations. An estimate has been made of the probable local flow from the motor-vehicle registration. Table 17 gives the actual and estimated travel for 1927 and shows the percentage that the actual travel is of the theoretical. From Table 19 and a knowledge of the particular locations and the progress of highway improvement it is possible to derive the percentage expectancy for the several sites.

TABLE 17.—PROBABLE LOCAL TRAFFIC FLOW OVER BRIDGES IN THE FIRST SERIES.

Bridge No.*	Traffic, 1927.	Estimated traffic, 1927.	Expectancy, in percentage of theoretical or estimated traffic, 1927.
1	717	1 000	72
2	210	425	50
3	120	300	40
4	40	150	26
5	600	750	80
6	25	125	20
8	155	325	48

* See Fig. 11.

Expected Income.—Having tabulated the probable travel for 1930, 1935, and 1940, it only remains to compute the income for each year modified by the expectancy as listed in Table 14. The specified toll charges are given in Table 15 and simple multiplication therefore produces the result shown in Table 18.

TABLE 18.—ESTIMATED DAILY AND ANNUAL TOLLS FROM BRIDGES IN THE FIRST SERIES.

Bridge	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 8.
1930.							
Tolls per day:							
Tennessee cars	$ 339	$ 312	$ 205	$ 130	$ 263	$ 47	$ 135
Foreign cars	130	115	43	11	32	6	11
Total per day	469	427	248	141	295	53	146
Percentage of expectancy	80	55	50	35	90	30	60
Probable daily	$ 374	$ 235	$ 124	$ 49	$ 265	$ 16	$ 88
Annual	136 000	85 500	45 200	18 000	96 500	5 800	32 000
1935.							
Tolls per day:							
Tennessee cars	$ 386	$ 341	$ 234	$ 150	$ 282	$ 53	$ 154
Foreign cars	195	172	86	23	81	10	19
Total per day	581	513	320	173	363	63	173
Percentage of expectancy	90	60	60	40	95	35	70
Probable daily	$ 523	$ 308	$ 192	$ 69	$ 344	$ 22	$ 121
Annual	191 000	112 000	70 000	25 200	125 000	8 000	44 000
1940.							
Tolls per day:							
Tennessee cars	$ 420	$ 370	$ 243	$ 162	$ 288	$ 55	$ 175
Foreign cars	243	230	143	29	130	13	26
Total per day	663	600	386	191	418	68	201
Percentage of expectancy	100	70	75	50	100	40	75
Probable daily	$ 663	$ 420	$ 290	$ 95	$ 418	$ 27	$ 150
Annual	242 000	153 000	106 000	84 800	152 000	9 900	55 000

TABLE 19.—ESTIMATED ANNUAL COST AND TIME REQUIRED FOR RETIREMENT OF BONDS ON BRIDGES OF THE FIRST SERIES.

Bridge No.	INCOME.			Cost.	Interest at 4½ per cent.	Upkeep and operation.	Annual cost.	Time to retire cost, in years.
	1930.	1935.	1940.					
1	$136 000	$190 000	$242 000	$800 000*	$36 000	$10 000	$46 000	7
2	85 500	112 000	153 000	1 014 359	45 600	10 000	55 600	13
3	45 200	70 000	106 000	850 513	38 300	10 000	48 300	15
4	18 000	25 200	34 800	727 445	32 800	10 000	42 800	Indefinite
5	96 500	125 000	152 000	214 323	9 600	9 000	18 600	3
6	5 800	8 000	9 900	62 104	2 800	5 000	7 800	25
8	32 000	44 000	55 000	155 481	7 000	5 000	12 000	6
Total...	$419 000	$575 200	$752 700	$3 824 225	$172 100	$59 000	$231 100	11

* Estimated cost.

All the bridges of this group have been let to contract. The costs for each structure and the cost for the whole group, therefore, are approximately known. The probable annual cost may be easily estimated. Table 19 gives the estimated annual cost and the probable period required to retire the cost of construction.

W. W. CROSBY,* M. AM. SOC. C. E. (by letter).—Concerted and noteworthy surveys to develop the relations between the use of roads and their design and upkeep were made prior to 1924. The Final Report of the Special Committee of the Society on Materials for Road Construction† shows that traffic was then recognized as "one of the most important" factors in the selection of the type of construction "considered from the standpoints of both economy and efficiency." In Section 4 of The American Highway Engineers Handbook, the even wider effects of traffic are pointed out, and considerable detail as to traffic censuses is included.

In the discussion‡ of a paper by Mr. H. R. A. Mallock, on "Construction and Wear of Roads", Mr. W. C. Copperthwaite has referred at some length to the wear as compared with traffic, citing London statistics of the latter, and many other contributors to the discussion recognized the relation of use to design and costs.

Reference to the *Proceedings* of the Association Internationale Permanente des Congrès de la Route will show that, since 1908 when this Association was organized, the relation of "use" to "costs" and other features of highway design and execution has been prominent in the minds of highway engineers and authorities all over the world.

Even Macadam and Telford were alert to the relation of "use" to "results" and, in his "Remarks on the Present System of Roadmaking" (1820), Macadam gave some suggestions as to the establishment of these relations.

Probably these scattered references to the extensive bibliography of traffic statistics will suffice to indicate the age of the consideration given to the subject.

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* Cons. Engr., Coronado, Calif.

† *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1388.

‡ *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVIII (1908–09), Pt. IV, p. 134.

The recent traffic surveys in Ohio are, of course, interesting, even if any novelties revealed by them are local, and confirm certain facts or principles indicated elsewhere. It is through what might be called the "integration" or "summation" of such local facts that a proper perspective for the scientific side can be obtained.

The great difficulty in the United States since the beginning of modern road work about 1895, has been that the use statistics concerning highways have always been behind, a kind of a "tail to the dog" of traffic development. Now, that more funds are available for the capture, examination, dissection, or analysis of this "tail", its importance is attracting attention from even others than highway engineers.

In some quarters there seems to be a tendency to make "the tail wag the dog"; that is, some argue that all features of a highway design should be based solely on actual or estimated traffic figures. The writer still argues that, of course, proper design demands consideration of traffic data, but he contends, "in and out of season", that after such consideration should come thought for perhaps more immaterial things. As the standards of living rise, pleasure and comfort make their demands for consideration, in addition to those of economy.

Charles Randolph Thomas, Jr.,* M. Am. Soc. C. E. (by letter).—The analysis made by Mr. Schlesinger shows rather clearly the importance of a study of population figures in planning roads. "The volume of highway traffic in an area is largely a function of the population of the area," as stated and proved by the author for Ohio, is also a fundamental proposition laid down by the late A. M. Wellington, M. Am. Soc. C. E., as applied to railroads. The development of modern highway traffic is producing conditions that are more and more like those that apply to railroads and telephone companies. Similar methods of analysis, which have reached a high degree of refinement in railroad and telephone traffic studies, may be applied with some degree of success to an analysis of highway conditions.

For purposes of predicting future traffic on a route, the great value of a traffic survey seems to be that it establishes the fact that highway traffic in general varies with population. In other respects, a traffic survey is like "locking the stable door after the horse is gone." The traffic is already there. Just how much traffic may be expected and what set of conditions will produce a given traffic is the information that the projector of a highway wants.

In cases where the traffic did not seem to vary so exactly with the population on the State roads, the author states that other roads carried the traffic. Might it not also be assumed from the same data that the economic location of the State road was poorly made—the road was not in the right place?

Another interesting conclusion is that only 14% of the traffic exceeded a 50-mile limit of trip—through traffic is relatively small. An analysis of traffic from the standpoint of the area served would seem to be possible, and such an analysis should give interesting and useful information, especially in predicting traffic.

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* Editor, *Good Roads*, Chicago, Ill.

The determination of the potential traffic in the area would give maximum figures useful in designing. The variable of "attraction of traffic" is eliminated by assuming that the road properly improved will get all the traffic, much of which is now going by other routes. This, of course, applies only to the local 50-mile haul traffic, which in Ohio is 86% of the total.

The subject is one that might be discussed and elaborated at length with interesting results. Suffice it to say that the old railroad principle of location holds true, namely, that there is one best route between two points or a group of points; that this route should be found and improvement made in proportion to traffic to be anticipated when the traffic area is developed to its full present possibilities; and if funds are available it may be wise to anticipate future traffic possibilities of the area.

The economic study of highways as discussed by Mr. Schlesinger also involves: (1) The political and social aspects and the effect of taxation and intangible benefits; (2) the more easily measured costs of construction and maintenance and the adjustment of such costs to individual vehicle operating costs; and (3) the monetary savings or profits, if any, from improved transportation facilities.

By limiting the subject to highways constructed by private enterprise under reasonable charters from the State, many social and political aspects are eliminated. This permits a study of the subject from the standpoint of commercial success or failure as measured by the yardstick of business—money profit or loss.

It is the purpose of this discussion to review briefly the development of old and modern toll roads and to present a method of studying certain of the economic factors to be considered by the engineer in making a survey of a proposed toll-road project under modern conditions, so that he may determine within reasonable limits of error the relative profit or loss of the project.

Toll Roads in the United States and England.—Immediately after the American Revolutionary War there was an insistent demand for improved roads, but the States and counties were too poor and the people would not tolerate excessive taxes for road building. Therefore, the English method of financing and building roads was adopted. The various States granted to private companies the right to build roads and charge tolls for their use.

The fundamental principle of this turnpike system was to transfer the cost of road building from the State to the users of the road. In the beginning only the main roads were to be turnpikes. The weakness of the turnpike system was that there grew up an endless number of isolated companies frequently under-financed and operating in a territory in which the volume of traffic would not permit an adequate return on the investment, even with tolls amounting to all that the traffic would bear. (A similar situation existed later with respect to some railroads). These private turnpikes were so short that the traveler was annoyed continually by the obstruction of gates as he passed from the road of one company to that of another.

Conditions in England under this form of turnpike system have been described by Mr. Edwin A. Pratt.* He states that when the need arose for better roads in any district, a group of land owners and others would apply to Parliament for authority to raise a loan to be used for improvements. They would then build or repair a road and set up gates or bars at which they could exact tolls for the payment of debts and for maintenance charges. In theory, this was intended as a temporary arrangement and that after the first cost was paid the road would become free for public use. Charters were granted for limited periods of about twenty years and these were renewable if the debt remained unpaid at the end of the period. Mr. Pratt states that:

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* "History of Inland Transportation and Communication in England," by Edwin A. Pratt, p. 82.

> "* * * one result of the excessive localization of the turnpike system was that trusts of absurdly large proportions were created to look after absurdly small stretches of road."

The turnpike system, accordingly both in England and America, fell into disrepute; it was killed by mismanagement and lack of business due to railroad competition before the coming of the revolution in highway transportation which was brought about by the motor vehicle. At present (1929) there are only a few of the old-fashioned turnpike companies in existence.

Nevertheless, the era of road building, brought on by the introduction of private initiative, marked the beginning of a period of rapid expansion in general business due to improved transportation facilities both in America and England. This was the era of the great engineers, Macadam and Telford.

Under modern conditions, with highway traffic completely motorized, only a few roads have been built by private capital. The failures of the Nineteenth Century are still too fresh in the minds of the people, and there is a widespread prejudice against toll roads, in many cases without sound reasons.

Toll Roads in Italy.—The outstanding modern development in toll highways built to care for motor traffic is the "autostrade" system in Italy. It has been projected and partly developed under the engineering leadership of Pieoro Puricelli. His basic ideas on projecting these motor roads, as expressed in a communication to the writer, are as follows:*

1.—Motorways should supplement the ordinary highways rather than compete with them, and for this purpose should be designed for the greatest degree of efficiency consistent with the expected traffic.

2.—These roads have a special function and should be constructed only where peculiar local conditions of traffic, etc., indicate their advisability.

3.—Toll financing of motorways is not a characteristic necessity, but it is a legitimate means of obtaining income from a capital expenditure.

4.—In their projection, construction, and operation, motorways are comparable to railroads. They should be designed for express traffic in a class distinct from local highways.

5.—Under private enterprise, the selection of routes and the execution of a project are regulated by the presence or absence of factors that insure a satisfactory financial return. Political pressure is reduced. Motorways should be built only where traffic studies give promise of a sufficient income.

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* Also, see, "Motor Roads and the Milan Lakes," by Pieoro Puricelli, Autostrade, Milan

6.—Existing roads need not necessarily be abandoned. If they are improved they may serve as feeders to the express highways.

7.—The mere presence of poor roads that are the result of improper maintenance does not justify the construction of a motorway.

8.—Existing roads often do not meet the needs of motorways because they are narrow and winding, they have bad surfaces, sharp turns, concealed intersections, interurban traction lines along the side, grade crossings, and inadequate traffic control. They are inclined to pass frequently through congested areas, and they are subject to the nuisances created by dust. These conditions are most easily controlled by designing separate express motorways. It is estimated that the Italian Government will take over the present motorway system before 1980.

The Motorway from Milan to the Italian Lakes Region.—This 60-mile highway was begun in 1923 and placed in operation in 1925. It is controlled by a limited liability company and cost approximately $75 000 per mile. The obligations were issued by the company and the Government guarantees the payment of interest, in return for which it takes over the control of the road in 1973.

In 1926, the receipts and expenditures were as follows:

Receipts:	
Trip tickets	$185 000
Season tickets	30 000
Advertising space	25 000
Total receipts	$240 000
Expenditures:	
Operation and maintenance	$ 80 000
Sinking fund and loans	72 500
Amortization of capital	12 500
Interest charges	70 000
Total expenditures	$235 000

The balance sheet thus showed a net profit of $5 000. There was an average of 700 vehicles per day of all kinds using the motorway during 1926 and in 1927 the daily average was 900. During the first months of 1928 the average was greatly increased.

Tolls.—The toll schedule in Table 20 was in effect for two years, after which a 20% reduction was adopted. Subscription cars are good for one year over the entire system, and provision is made for companies owning several vehicles; that is, a 20% reduction for two vehicles, 25% for three, and 30% for four or more. Public service motor vehicles are not issued subscription cards and are charged triple tolls. Vehicles with solid rubber tires pay double toll.

Entrepreneur Studies.—The inception of a toll-road project presents certain speculative questions as to the profits possible from the investment of capital. These may be listed somewhat as follows:

1.—Will the financing, construction, maintenance, and operation costs permit income with a sufficient margin to provide for dividends and the retirement of the capital invested?

2.—What are the sources of present and future traffic and how productive of income traffic will be these sources?

3.—What are the details of immediate and future construction and operating expense on the most economical route and just where should this route be located?

4.—What is the proper investment to make so that traffic will be handled most economically and, at the same time, produce the greatest net revenue?

5.—What kind of franchise may be obtained from the State or local authorities and what are the conditions of the franchise, such as duration, recapture, renewal, etc., that must be met?

6.—What measures will be necessary to attract traffic to the highway and what will be the cost of a measure such as advertising?

TABLE 20.—Tolls Paid on Milan-Lakes Motorway.

Type of vehicle.	Ticket.				Fifty coupons.	Subscription.		
	Complete Journey.		Short Journey.			Annual.	Half year.	Quarterly.
	Single.	Return.	Single.	Return.				
Small vehicles......	$0.50	$0.75	$0.30	$0.50	$17.50	$45.00	$38.00	$30.00
Cars and trucks, to 17 h. p. trailers....	0.75	1.00	0.40	0.65	25.00	67.50	56.00	45.00
Cars and trucks 17 to 26 h. p...........	1.00	1.50	0.60	0.90	33.75	90.00	75.00	60.00
Cars and trucks of more than 26 h. p..	1.50	1.90	0.80	1.10	42.50	112.00	94.00	75.00
Buses and cars, to 20 seats...............	2.50	3.75	1.60	1.90	......	......	......	......
Buses and cars of more than 20 seats.	3.75	5.60	2.40	3.30	......	......	......	......

To answer some of the questions is the mission of the engineer employed to make a preliminary report; the answer to others has an element of speculation which will be discovered only when the highway has been in operation over a period of years.

Scope of Preliminary Engineering Report.—The scope of the preliminary engineering report suggested is limited by the questions asked by the entrepreneurs as to the feasibility of the project, and by the time and funds available for the investigation.

Two basic ideas must be kept in mind in making such a report: First, that a motor vehicle has freedom of movement and will find the economic route that offers convenience, speed, and economy; and, second, that the best location between maximum traffic sources must be found and there must be placed on that location the type of road (pavement, signaling, crossing, etc.) demanded by traffic, with due regard to future traffic at the end of a term of years.

The report might well include: (*a*) A traffic reconnaissance of the area through which several possible ground locations might be made; (*b*) estimated traffic from various sources on the approximate paper location made from a study of the area, consideration being given to possible alternate routes at certain points; (*c*) an estimate of revenues both present and possible in the future, as indicated by the population and industrial studies made of the area tributary to the route; (*d*) estimates of construction and maintenance costs; (*e*) estimates of operating costs; and (*f*) summary of financial and charter requirements.

Conclusions and recommendations as to the feasibility of the project might well be summarized at the beginning of the report.

Traffic Studies of the Area.—The study of traffic within the area tributary to several possible locations in the general direction thought feasible for the projected highway is limited. Mr. Schlesinger has found that, over large areas such as a State or parts of a State, the traffic on the roads varies directly as the population. The accumulation of traffic on a railroad route was found by Wellington* to vary as the square of the number of traffic sources, and it is believed that for limited areas the traffic may develop in even a larger ratio on highways.

The author states that the ratio of through traffic (trips of more than 50 miles) was approximately 30% in Ohio; that bus traffic was 1.5% of the total passenger traffic; and that trucks were 9.5% of the total traffic on the State system.

In a study of comparative rail and highway tonnage made in Ohio in 1925† the proportion of motor-truck tonnage decreased with the length of haul, as shown in Table 21. However, with the improvement in motor vehicles and methods of operation the length of profitable haul is gradually increasing.

The factors entering into a determination of the traffic in the area have been summarized elsewhere by the author as follows:‡

> "The formulation of a method for approximating the traffic on a given highway without the necessity of a field survey is within the range of possibility, using the data available as a result of transport surveys. It is believed that the problem is capable of solution if proper consideration is given to such factors as:
>
> 1.—The tributary area.
> 2.—Population density.
> 3.—Motor vehicular registration.
> 4.—Character of population—urban or rural.
> 5.—Classification of predominating industry—industrial or agricultural."

Maximum traffic will, of course, be the sum of the potential traffic of the area and the foreign traffic coming into the area. However, it is scarcely possible that this maximum can ever be attained.

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* "The Economical Theory of Railroad Location," p. 713.

† Traffic Survey by U. S. Bureau of Public Roads and Ohio Highway Department, reported in *Roads and Streets*, October, 1927, p. 427.

‡ Extract from an address before the American Association of State Highway Officials, 1926.

Traffic Reconnaissance of the Projected Line.—Traffic studies of the projected location of a route may be conducted in sections covering a width approximately equal to the normal maximum trip of a motor vehicle.

The comparative weight to allow in securing local traffic for the road; the sacrifices of distance necessary in order to reach certain traffic sources that accumulate traffic; how near to run to cities and towns; the distance gains made in by-passes; and the intermediate changes in alignment to shorten the route, are all matters that must be considered as affecting the total volume of traffic. While the traffic on the road is increased by lengthening the line, if that lengthening is too great drivers become discouraged, the road possibly becomes too congested for the service rendered, and traffic will seek new routes. The effect of a secondary road on the traffic on the main road also must be considered.

TABLE 21.—PROPORTION OF MOTOR TRUCK AND RAILROAD NET TONNAGE ACCORDING TO LENGTH OF HAUL FOR AVERAGE MONTH, 1925.*

Length of haul, in highway miles.	MOTOR TRUCK.		RAIL (CARLOAD).		RAIL (LESS THAN CARLOAD).		TOTAL.	
	Tons.	Percentage of total truck and railroad tonnage.	Tons.	Percentage of total truck and railroad tonnage.	Tons.	Percentage of total truck and railroad tonnage.	Tons.	Percentage of total truck and railroad tonnage.
Less than 20	6 091	84.5	1 112	15.4	10	0.1	7 213	100.0
20 to 30	5 973	54.7	4 803	44.0	145	1.3	10 921	100.0
40 to 59	2 299	32.0	4 484	62.4	404	5.6	7 187	100.0
60 to 99	980	24.2	2 409	59.4	663	16.4	4 052	100.0
100 and more	157	2.3	5 280	77.4	1 353	20.3	6 820	100.0

* Based on tonnage between Columbus, Ohio, and thirty cities having rail connections, report of U. S. Bureau of Public Roads.

It should be remembered that no single road is completely independent of other competing roads, and when congestion comes there is a tendency for the traffic to "spill over" to competing routes. However, traffic has a tendency to concentrate on certain routes that afford convenience, speed, and economy.

Factors That Affect Traffic.—The function of a highway is to provide a smooth, convenient route from the point of freight or passenger departure to the destination, and it must not be assumed that traffic will come to a road just because it happens to be a good road. Other factors enter the problem.

Certain roads acquire a name for being good roads and get the traffic apparently for no better reason than the Biblical allusion, "to him that hath shall be given". Likewise, the saying, "give a dog a bad name and it will stay with him", applies as well to highways and possibly explains why some roads lose traffic.

An apparently inconsequential matter that causes a road to miss a town may result in a loss of traffic that is out of all proportion to the advantage gained. The loss of traffic due to missing a town at a distance great enough to make access to the road difficult must be balanced by the traffic gained by the other, perhaps shorter, route.

Wellington states that "errors in the original laying out of a route, unlike errors in subsequent management, are mainly irremediable—a kind of fixed charge for folly forever". He also estimates that the loss of traffic* varies from 10 to 25% of the total traffic to be obtained by going directly through the center of the town for each mile a railroad passes at a distance from the center of the town. These figures, of course, apply only to railroads, but they give a guide as to a method of computing the effect of moving a highway away from a town.

The factors that cause variations in the location of a route are: Manufacturing towns that originate a large amount of traffic; special industries that must be served; the existence of competing routes; and special reasons that arise, and are peculiar to the existing conditions. An attempt to pass through the center of gravity of the population of two towns by going between them often leads to fallacious results unless more than two towns are considered.

Methods for reconnaissance surveys have not appeared in print so far as the writer has been able to determine. The method outlined herein is suggested as a procedure to be followed only as conditions justify.

The first step in a traffic reconnaissance survey is the collection of all available data in the form of published U. S. Geological Survey maps to be used in making notes of data and all reports as to motor-vehicle registration in the countries to be traversed. Reports showing the location of industrial plants, tonnage of farm crops marked at various cities, census figures as to population of counties, and similar material, also will be useful. The various civic chambers of commerce can furnish valuable information.

Since it may be roughly estimated that about 85% of the motor-vehicle travel will not go more than 50 miles on a single trip, and about 70% will not go more than 30 miles, it may safely be assumed in general that a margin of 50 miles, 25 miles on either side of the proposed line, will be sufficient to embrace the traffic that will be attracted to the road.

The next step is to make a careful study of the 50-mile strip of territory, estimating the tonnage to be hauled. This tonnage can easily be converted into the approximate number of motor-vehicle trips necessary to transport. To estimate pleasure car traffic the registration in the area may be multiplied by about 5 000 miles of annual use per year for each car, and this, in turn, may be changed to trips past a certain point with a suitable correction. Through traffic may be estimated from traffic counts at selected points, suitable corrections being made for seasonal variations and week-end concentrations.

The third step is to post men at selected points, determined by a study of maps and personal examination of the roads, and to inquire about traffic and such matters as the possible sale of filling stations or revenue from other similar sources. The men should count the traffic on selected days of the week covering 24 hours each day.

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* "The Economical Theory of Railroad Location," p. 63.

From the traffic counts and the other data available, a rough approximation as to existing traffic can be obtained. This estimate can be corrected for attraction of traffic to the new road and the influence of competing roads. To determine future traffic, the potential traffic in the area based on tonnage hauled and registration predicted, as well as population increase, can be estimated as the maximum regional traffic. The determination of increase in through traffic is not so simple, but may be estimated from increased registration in the State, by considering that through traffic, in normal cases, is not more than 25% of the total traffic.

Future traffic may be estimated from the slope of the curves of traffic by years, especially the curve of motor-vehicle registration in this area. Predictions more than five years in advance are likely to be very misleading.

Evaluating Distance and Time.—The saving of shortened distance to transportation agencies is readily determined by a simple computation using the method of W. H. Connell, M. Am. Soc. C. E.* For example, with an operating cost of about 10 cents per mile and a traffic consisting of 85% light motor vehicles and 15% trucks, the cost for each 1 000 ft. over a paved road, or all-weather road, is about 2 cents for each vehicle. For 1 000 vehicles per day the cost would be about $20, or $7 300 per year. The saving per mile per year due to shortening the route is, therefore, approximately $38 700, or $774 000 over a period of 20 years, the life of many bond issues.

The value of a car-minute as determined for congested areas adjacent to New York City by the State of New Jersey in studies of highways was: For trucks, 2.3 cents; for buses, 2.1 cents; and for pleasure cars, 1.0 cents. In applying these figures to single grade crossings† it was found that on a highway carrying four lines of traffic averaging 20 000 000 vehicles per year there was a loss of 7 000 000 car-minutes, or $154 000 per year which, capitalized at 4%, equals $3 850 000, the sum that might reasonably be invested to eliminate this delay.

Location, Curves, and Grades.—The effect of location, curves, and grades on the first cost is direct and must be considered from the standpoint of the initial investment in the road—whether it is desired to build a cheap or expensive line.

Having determined from the consideration of the various traffic surveys, the traffic-source points through which the projected line must pass, the problem becomes one of finding the straightest and most economical line through these points. This line is located on the ground, by field surveys of as many routes as seem feasible, using the methods familiar to highway engineers.

The elimination of grade crossings with railroads and intersecting highways is one of the largest items of expense and must be considered carefully because of the liability of a private corporation to damages for accidents occurring on its property. By observing care in locating the intersections of highways so that they meet at different levels the cost of grade separation can be reduced. On level ground a treatment of the type shown in the sketch, Fig. 13, may be adopted. It is desirable to have as few intersecting highways as possible for safety considerations, but it is also desirable to have as many traffic sources as possible from the standpoint of local revenue. The private drive nuisance on public roads, which permits the entry of vehicles on the road at practically any point, is eliminated when the road is on a protected right of way with sharply defined places of entry and departure for traffic.

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* Report of Investigations of Paving and General Highway Conditions by the Commission Appointed by the National Paving Brick Manufacturers Association, p. 19.

† "Economic Theory of Highway Location," by F. Lavis, M. Am. Soc. C. E., *Roads and Streets*, October, 1927, p. 443.

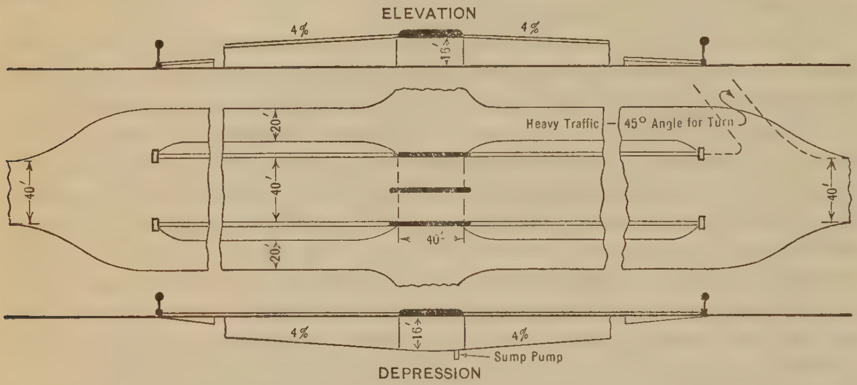

FIG. 13.—PLAN FOR HIGHWAY GRADE SEPARATION ON LEVEL GROUND BY EITHER ELEVATION OR DEPRESSION OF ONE ROADWAY.

The saving in time by the elimination of delays at crossings and other points of congestion will be an important factor in making the road more attractive to traffic, thereby increasing revenue. The capacity of the road is also increased by the time saving.

Signaling for traffic control while entering or leaving the road and en route should be reduced to a maximum necessary for safety, in order to avoid delays and annoyance for the traveler. The amount expended on signaling will vary with the number of danger points on the road.

Estimating Revenue.—Revenue to a toll road may ordinarily accrue from (1) use charges or tolls for various types of vehicles; (2) advertising space and concessions leased; and (3) rent of surplus land usually obtained in acquiring right of way.

Toll charges for various types of vehicles should be applied as simply as possible. In Italy, various categories based on the horse-power of the vehicles are in use; term or season tickets are issued, in addition to ordinary trip tickets.

The value of connecting routes as sources of both local and through traffic is self-evident. These routes also intersect other roads, in most cases, and one of the problems of the toll highway officials is to influence this traffic to use the toll roads because of the superior advantages offered. There also exists a competition with other roads that parallel the toll highway and here, again, a problem is evident.

From these and other factors it is apparent that the chief problem from the standpoint of revenue is the sale of transportation facilities which may have sufficient advantages over other available facilities so that the traffic is willing to pay a premium to enjoy these advantages.

The sale of transportation, like the sale of anything else, is based on making the user of the facility want it, and the same effort and technique must be employed as in the sale of any other commodity. In this sales effort, advertising plays an important part. Advertising campaigns based on the recommended procedure of specialists in such work are advisable and undoubtedly will have an important effect in increasing the travel on the road.

Some revenue will be derived from the leasing of billboard space on the right of way, but it has the disadvantage of obstructing the view of the landscape and makes the highway less attractive to the traveler. Concessions such as gasoline filling stations, telephones, refreshment stands, etc., are also a sure source of revenue.

The leasing of excess land owned by the company may be a source of revenue which, in some cases, may be used to offset the carrying charges of such land. It is inevitable that in acquiring right of way, whether by easement or purchase, there will be much of it not used for highway purposes. With the construction of the road this land will increase in value in most cases and may be sold later at a profit.

The ratio of local traffic (not more than 50-mile trip) as determined by traffic surveys in several parts of the United States, is from 10 to 25% of through traffic varying with the season, the section of the road, and the section of the country. It is quite evident, therefore, that the bulk of the revenue traffic will be local in nature and, at present, will be for the most part light pleasure automobiles. It is highly probable, however, that a decided increase in truck traffic will be seen within the next decade.

Construction Costs.—These costs, of course, depend much on the investment in roadway and protective works which the study of present and future traffic indicates will be profitable for the location selected. In determining the first cost of a pavement, the relative proportion of that cost necessary for maintenance must be considered as well as that of renewals at the end of its useful life.

Financial Study of a Toll Road.—For purposes of outlining a method of studying the working of a proposed toll-road project, consider a hypothetical case.

A toll highway is proposed to connect two cities of about 1 000 000 population each, that are about 80 miles apart. Between these cities are ten smaller cities varying in population from 5 000 to 50 000 which will be by-passed but tapped with short branch roads. There are ten entry points to the highway at the small cities and also ten other minor intersection points of industrial centers at which entry may also be made, making a total of twenty-two entry points, including the two terminals. Initial traffic is approximated at 4 000 miscellaneous vehicles per 24-hour day.

The investment in the project is estimated roughly, as follows:

80 miles of operating highway at $100 000	$8 000 000
Working capital stock issued as needed	2 000 000
Total	$10 000 000

The items that may be considered in the construction costs and the percentage of the total for each item for a road constructed at a cost of $100 000 per mile may be approximated, as follows:

	Estimated cost.	Percentage of total costs.
Right of way	$12 000	12
Grading and draining	15 000	15
Bridges	15 000	15
Crossing eliminations	25 000	25
Pavement	25 000	25
Miscellaneous protective works	5 000	5
Signaling and lights	2 000	2
Buildings	1 000	1
Total	$100 000	100

A statement of the annual income and expense of the report might be approximately, as follows:

Income.—	
Through traffic tolls, 1 000 vehicles per day at $1.00	$ 365 000
Local traffic tolls, 3 000 vehicles per day at 50 cents	550 000
Season tickets, 2 000 at $50.00	100 000
Advertising space leased	50 000
Concessions leased, filling stations, etc	50 000
Total	$1 115 000
Expense.—	
Fixed Charges:	
Interest on bonded debt of $8 000 000 at 6%	$ 480 000
Sinking fund to retire $8 000 000 in 50 years, amount annually at 5% interest	39 000
Operating expense, annual (see statement)	450 000
Proposed dividends on $2 000 000 stock at 6%	120 000
Surplus, annual	26 000
Total	$1 115 000

Operating Expense Statement.—

		Estimated cost.	Percentage of total expense.
Maintenance of Pavement and Bridges:			
80 miles at $300	$24 000		
Miscellaneous at $50	4 000	$28 000	6
Renewal of Pavement and Bridges (20-Year Life):			
80 miles, $40 000 mile cost at 5% interest, per year		97 000	22
Maintenance of Protective Works:			
80 miles at $50		4 000	1
Renewal of Miscellaneous Works (10-Year Life):			
80 miles, $6 300 mile cost at 5% interest, per year		40 000	9
Collection of Tolls:			
30 men at $2 000 per year		60 000	13

Operating Expense Statement (Continued).—

		Estimated cost.	Percentage of total expense.
Policing (Motorcycle):			
4 men at $2 500 per year		$10 000	2
Executive and Clerical:			
Manager	$15 000		
Engineers	10 000		
Clerks	4 000		
Accountants	6 000		
Miscellaneous	$5 000	40 000	9
Damage claims and legal		15 000	3
Advertising and sales		20 000	5
Taxes		50 000	11
Insurance		8 000	2
Miscellaneous and contingent		78 000	17
Total		$450 000	100

G. F. Schlesinger,* M. Am. Soc. C. E. (by letter).—The discussion of this paper has been of an informative and supplemental nature, as the subject-matter was not such as to call forth comment of a controversial character. Highway engineers and authorities are agreed that the information supplied by a comprehensive transport survey is essential to the proper planning of a program for future improvements.

Mr. Simpson calls attention to the impracticability of securing as complete information in a city traffic survey as in the Ohio survey of rural highways, and questions the necessity of what might be termed non-essential data in solving problems arising from city traffic. As stated by the writer some of the information made available by the Ohio Transport Survey "might be classified as being outside the scope of practicality". However the data were obtained at very little additional expense and, without discussing their practical utility, it may be stated that they have been a valuable contribution to the general knowledge of the fundamental principles of highway traffic. It has occurred to the writer that a knowledge of the origin and destination of traffic would be very desirable in a city where conditions require the designation of a system of "one-way" streets.

It is somewhat surprising that, according to the Columbus survey, the proportion of traffic in the main city streets should agree so closely with that on the main Ohio rural highways (the State System). It would seem logical that the proportion on the main city streets, as compared to rural routes, would be less, due to the following causes:

(1) The greater availability of alternate secondary city streets;
(2) The better condition of the pavements on alternate secondary streets; and,
(3) The tendency of city traffic to avoid congested and traffic control (manual and automatic) conditions on the main thoroughfares.

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It would follow that there must be other counteracting factors in the case of rural main highway traffic. There is probably a greater percentage of pleasure traffic outside the city, that does not follow the most direct routes. The fact that the maximum daily traffic occurs on Sunday in the country and that it is a minimum on that day in the city, according to Mr. Simpson, shows clearly the use many city motorists make of their cars over the week-end.

In his discussion, Mr. Van Duzer mentions the importance of careful methods in forecasting future trends and also the securing of additional data that later might modify the general plan based on the original survey. There should be periodic "follow-up" surveys, which can be conducted on a much smaller scale. He also suggests the possible utilization of highway transport surveys in connection with the safe operation and control of traffic, with which, he states, future highway administration may be expected to concern itself. The operation of the State highways has been placed, to a considerable extent, in the hands of the State highway engineering executives, and properly so. Such duties as the erection of route markers and warning signs belong to them, and, in some cases, the highway police force is under their control. However, in the cities, the control of traffic is usually under the public safety, police, traffic, or other department without engineering administration and, in most cases, without engineering advice.

Mr. Thomas draws an analogy between highway and railroad traffic. There is, however, a decided difference in respect to the composition of passenger traffic in the two types of transportation. The Ohio survey shows that 44.6% of the passenger-car traffic on the State Highway System measured in passenger-car miles is non-business or pleasure traffic, while railroad traffic is more predominantly of the business variety. Railroad passenger traffic, also, is not carried in small mobile units subject in routing to the almost instant direction of a relative large number of individual operators. For this reason it would not be accurate to assume that a "road properly improved will get all the traffic, much of which is now going by other routes".

Mr. Thomas quotes from the original paper to the effect that 14% of the traffic exceeded a 50-mile limit of trip. This figure applies to truck traffic and approximately 30% of all traffic exceeded 50 miles per trip. However, Mr. Thomas is correct in his conclusion that highway traffic is primarily a function of local transportation and of the population of local areas tributary to the highway. The writer agrees that the formulation of a method for approximating the traffic on a given or projected highway route without the necessity of a field survey is within the range of possibility, using the data and principles established as a result of transport surveys already made.

Some of the principles established by the traffic studies in Ohio, and by other comprehensive surveys, have been applied by Mr. Thomas to the problem of determining the probable commercial success of a proposed toll-road project. His analysis, suggested method of procedure, and application to a hypothetical case are admirable. Nor can the writer offer any valid objections to such private ventures in the transportation field, if they appeal to capital as profitable enterprises, assuming that the public interest receives the usual legal protection incident to quasi-public utilities. However, there is such decided opposition to toll roads in the United States that there is little probability of such enterprises being legally authorized until there is a great change in public sentiment. In the writer's opinion no legislative body, at the present time, could be persuaded to grant a toll-road company the right of eminent domain to condemn private property for the right of way required for such a project. While the gasoline tax is essentially a form of tolls, it has been termed a "painless tax" because of its method of collection. The motorist is not confronted with a barrier that will be closed to him until he has purchased a ticket. A projected toll road, as Mr. Thomas observes, would offer an excellent opportunity for the application of the principles of highway traffic flow in a purely economic study with social and political considerations eliminated. It is believed that in the future development of highway transportation in this country, as in the past, highway engineers will be required to deal with such social and political problems along with those of a technical nature.

That methods of approximating traffic are not only possible, but actually have been applied, is revealed in Professor Dougherty's description of the surveys in connection with proposed toll-bridge projects in Tennessee. The statement that "a 12-hour count taken in August, therefore, approximates the average 24-hour flow throughout the year", may give readers the impression—as it did the writer at first reading—that one 12-hour count during that month is all that is required for this approximation. The reference is to the average of daily 12-hour daylight counts taken during the month of August. The details of the analysis of probable traffic and consequent tolls are very logical and should be useful to those confronted with similar problems. The writer will inject the observation that the prevailing method of financing highway improvements by means of a tax on gasoline is essentially one of tolls.

Colonel Crosby calls attention to the fact that for a century or more highway engineers have recognized the utility of traffic data as factors in the design of roads and the selection of types of pavements. The writer believes that a more important and valuable utilization is in the establishment of a State-wide or regional plan for a future program of highway development. He agrees with Colonel Crosby that there is a benefit to society in the improving of highways that is indefinable and incapable of measurement in terms of traffic accommodation, or any other material units.

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Paper No. 1737

THE ENGINEER'S PART IN MAKING THE HIGHWAY SAFE*

BY A. H. HINKLE,† M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. E. W. JAMES, LOUIS P. BLUM, C. N. HAGGART, H. J. KIRK, W. W. CROSBY, VICTOR A. ENDERSBY, AND G. C. DILLMAN.

It is difficult to draw an exact line as to where the engineer's responsibility ceases with reference to safety on highways. Because the field is rather broad what is said here should not be questioned with reference to the exact limit of responsibility between any two classes of individuals.

Study shows that there are many contributory causes of accidents on highways, although any one accident may have its major source. For example, on a rainy day with the windshield blurred, a driver may have approached, at high speed, a sharp curve having no superelevation, where there were leaves on the wet surface, resulting in his machine skidding off the pavement. It is seen that six different factors may have contributed to the accident. Too frequently the driver of a machine that has been in an accident does not himself realize all the factors. This, in itself, may handicap him in profiting by his past experiences.

For discussion, the causes of accidents may be classified as follows: (1) Incidental to the operator himself, such as bad vision, sleepiness, dull sense, over-work, excessive speed, general disregard for traffic rules, etc.; (2) defects in the automobile, such as imperfect brakes, obscured vision due to poor curtains or large corner posts, glaring or too weak headlights, broken connecting rod, etc.; (3) highway conditions, such as sharp curves, steep grades, slippery surfaces, high crowns, narrow pavements or shoulders, steep ditch slopes, narrow bridges, holes in the surface, lack of superelevation, railroad grade crossing, etc.

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The third class of accidents is primarily a problem for the highway engineer. About 30% of all the accidents recorded in Indiana in 1926 were due to highway conditions. It would seem that in a measure such statistics would show the degree of responsibility to be taken by the engineer. While limited finances and other restrictions must of course assume their share of responsibility for the engineer's acts, yet he must offer the proper advice to those to whom he reports. Hence, the engineer is primarily responsible for the proper design and upkeep of the highway so as to reduce to a minimum the conditions that contribute to accidents.

CURVES

Vision.—A minimum clear sight of 500 ft. between any two points 5 ft. above the road surface seems to have been generally adopted for both horizontal and vertical curves. While this might be recognized as a desirable standard, it should ever be kept in mind that different conditions will affect the safety features connected with the sight distance, so that a clear sight of 400 ft. in one location may be no more dangerous than one of 600 ft. in another. For example, if the point in question is at the end of a long tangent in open country, where a high speed is normally invited, a short sight distance becomes far more dangerous than it would if it were closely bounded on either side by curves of lower degree which would normally slow down the speed of a vehicle. Also, in built-up districts where the speed is normally much lower than in open country, a shorter sight distance is permissible.

Horizontal Curves.—In open country where a high speed may be expected, it is desirable from a safety point of view, to limit curves to 5 or 6 degrees. Due to the continually higher speeds at which vehicles travel, it is best where the cost is not great to keep the sharpness of the curve several degrees lower than that considered dangerous at present, thus avoiding what may be a demand in later years to reconstruct on a flatter curve. In mountainous country, and in other strategic locations, it is necessary from an economic point of view to construct much sharper curves; but this may be done in such a way that the danger is not necessarily great. Due to the natural tendency of drivers to reduce speed on a curve, a 20° curve in one location may be no more dangerous than a 6° curve in another. The driver is bound to be more cautious in hilly country, full of curves and grades, than at the end of a long tangent with an unexpected curve.

Perhaps the better way would be to limit the curve to not more than, say, 6° sharper than any preceding curve ending within 200 ft. of it. From a safety point of view, small curvature, or small changes between adjacent curves, regardless of degree, may not necessarily be dangerous. In fact, due to the greater caution naturally taken by drivers on a curved line, it is possible that with the proper design an irregular line with low degree curves or a low differential may be safer than the average tangent. Statistics of accidents seem to show this to be true. Danger is a factor of speed, other things being equal, and, therefore, anything that contributes to a lower speed may result in fewer accidents.

Precaution should be taken to guard against using the maximum degree of curve where it is not necessary. In many cases where there is a 10° curve, a 5° curve could have been used just as well and even more economically because of the saving in pavement.

Superelevation.—Curves of 2° or more should be superelevated. This generally may be computed from the formula:

$$e = \frac{V^2}{15\ R}$$

in which,

e = the superelevation, in feet per foot of width;
V = the velocity, in miles per hour; and
R = the radius of curve, in feet.

In this formula, the speed is assumed at 30 miles per hour, with the maximum superelevation as 0.1 ft. per ft. of width.

The slight difficulty due to a vehicle slipping toward the inside of a curve on an icy surface, may be overcome by building a comparatively wide inner shoulder at such places at the normal shoulder slope. The outside shoulder for its full width should take the same superelevation slope as the pavement. On superelevated curves the crown of the road should be reduced to zero at the point of maximum superelevation.

Widening on Curves.—It is well to widen the pavement on all curves of 5° or sharper. The widening should be on the inside and the width, in feet, should be about 0.3 times the sharpness of the curve, in degrees.

Due to the added cost, the widening on concrete pavements is frequently omitted on a curve of less than 8 or 10 degrees. This may be justified on light traveled roads. In such cases it is necessary to give more care to the inside shoulder by maintaining an added width of stone or gravel that will prevent a rut from forming along the pavement. As an alternate to widening the curve, a spiraled curve is sometimes used. The spiraling is not a proper substitute, however, for widening on high degree curves.

Widening the intersection of important roads has a great value in reducing accidents, perhaps as much in impressing the driver more emphatically with the fact that he is approaching an important and hence dangerous intersection as in increasing the sight distance or giving more room for dodging the other vehicle. At a certain right-angled intersection, one of the two heavily traveled highways is in a cut for about 100 ft. back. This intersection was widened at all four corners to a 100-ft. radius. Although accidents had been numerous previously, not one has been recorded in the two years since the highway was improved. It is interesting to note how drivers slow down as they observe the widened intersection. Unconsciously, the improvement seems to impress the mind of the driver that there is danger ahead.

GRADES

From a safety point of view it is desirable to keep long grades down to 3 or 4%, if no material additional cost is entailed. On main heavily traveled highways it is extremely desirable to limit all long grades to 5% even at considerable expense. On secondary roads, 7% may be taken as a desired maximum for long grades. For short distances, say, up to 300 ft., 7% grades may not necessarily be dangerous on the main highways, nor 9% grades on secondary highways. It should be kept in mind, however, that the same theory of relationship applies to grades as to curves. A steep grade will be much less dangerous if approaching lighter grades and curves have slowed down the speed. Thus, frequently, the steepest grades and sharpest curves are not the greatest source of danger.

Combined Grades and Curves

Care should be taken in combining maximum grades and curves. If each alone is a source of danger, their occurrence on the same stretch of road is sure to increase the hazard. Hence, the combination of the steepest grade and sharpest curve on any road should always be avoided.

It is difficult to determine what compensating value should be assigned to curvature in conjunction with maximum grade. Perhaps it is not far wrong to assume that 5° of curvature may be equivalent to at least 1% of grade. In hilly or mountainous country, it is well to guard against the mistake (frequently made) of securing long uniform grades at the expense of sharp curves.

Crown

The higher the crown the more dangerous is any pavement, especially on a rural highway where high speed prevails. Perhaps the effect is almost negligible up to 2 in. of crown in 18 ft. of width; however, on grades and curves any camber increases the danger to traffic. One should not be misled by the fact that the ordinary gravel and stone surface requires more crown on grades to prevent erosion of the surface and thus make the mistake of applying this same dangerous design to grades on pavements. Sometimes it is possible to carry the water down the slope on the pavement more cheaply than in the side ditches. In these cases the pavement itself should be widened 4 to 6 ft., at the same time narrowing the shoulder to a few feet with no side ditch, all water going down the pavement which should have curbs. This design in deep cuts will often save enough in excavation (to say nothing of future maintenance) to pay for the extra width of pavement. It necessitates spillways at the foot of the grade, but practically obviates the crown.

On a modern high type pavement there seems to be little excuse for using more than 1½ in. of crown in 18 ft. of width. A favorite highway warning sign is, "Dangerous When Wet". While the pavement may be on a light grade and of a type of surface which is more or less slippery when wet, the dangerous feature is greatly increased by the unnecessarily high crown. It may be difficult to prove absolutely the amount of additional danger due to the high crown; however, as there is no economy in such construction and little or no cost in eliminating it, the camber might just as well be kept to a minimum.

Width of Pavement and Road Shoulders

Exactly what effect the width of pavement may have on the safety of a highway is open to argument, because the wider the pavement, the higher the speed; and speed is a factor of danger. From observation, however, it seems that although the wider pavement may invite a higher speed, it affords at the same time a net contribution to the increased safety of the highway.

The accident statistics of the National Road leading east out of Indianapolis, Ind. (a 30-ft. pavement), as compared with those on the 18-ft. pavement going westward out of the same city, are instructive. The traffic on these two roads should be about the same. There seems to be no evidence proving the claim so frequently made that a three-lane pavement is more dangerous than one of two lanes. Out of the fifteen accidents reported on the 30-ft. pavement all but two happened at intersections due to traffic driving on the wide pavement from the side roads or private drives. No serious injuries or great damage occurred. On the other hand, on the 18-ft. pavement there was one fatality, three pedestrians were seriously injured, and twelve turn-overs or collisions occurred, in which the occupants received minor injuries. These meager records point to greater safety on the three-lane pavement.

Wide shoulders properly maintained serve traffic in an emergency. On the 18-ft. country pavements a 6-ft. shoulder has been used as a standard. On heavily traveled roads an 8-ft. width certainly is desirable, particularly where fills are made with a 1 on 1.5 slope. Not only do these shoulders serve traffic in an emergency, avoiding an accident, but also as a parking place for changing tires, or other reasons. Numerous accidents are traceable to cars parked on country pavements and most States have laws prohibiting such practice. However, unless sufficiently wide shoulders are provided the practice of parking on the pavement will be difficult to control.

Sufficient Width of Right of Way

While heavily traveled roads are requiring wider pavements and wider roadbed, it is also imperative to provide sufficiently wide rights of way. Every tabulation of accidents shows a growing number of collisions with telephone poles and other appurtenances. For any important country road, 60 ft. should be the minimum width of right of way, poles being placed at the outer edge. Heavily traveled highways where unusually wide pavements are necessary, should have the needed additional width. There is no excuse for sitting by and permitting people to be killed by running into obstructions that should have been properly located, not only for safety, but to provide a more artistic appearance and room for roadside planting. That many a heavily traveled road with lines of high poles close to the traveled way looks like an alley, is no credit to the engineer or other official who may be responsible for this condition. Many States have the practice of securing 80 and even 100 ft. as a standard for their main State roads. Such widths are not practical through rich agricultural lands, but in the sparsely settled, low-valued districts, they are commendable.

Slippery Pavement Surfaces

It is well known that a slippery road surface contributes to accidents. One section of wood block pavement (about four squares) was so slippery, due to the method of treatment, that one or more accidents occurred nearly every time the surface was wet; it was not uncommon to find three or four cars piled up after the beginning of a rain. Concrete and bituminous road surfaces are the most common pavements. Of the bituminous type, the bituminous macadam or surface-treated road is the most frequent in the rural districts. This type of road can be made into a less slippery surface by the proper grade and amount of bituminous treatment and of aggregate used for covering. On steep grades it is particularly desirable to spread a coating of 1-in. or 1¼-in. stone during the last hot weather of the season.

Some concrete road surfaces are far more slippery than others. It is extremely desirable that some research work be carried on to determine what might be done to make the best non-skid type of surface for this pavement. A dry mix and proper finish may be used to reduce the slipperiness to a minimum. It is well known that the so-called "harsh mix" of concrete gives the opposite of the slippery surface sometimes found. To what extent this mix can be used without materially lowering the strength of the concrete should be determined, especially for grades where slipperiness is a big factor.

The oil dropped from motors adds greatly to the accident hazard. A wider pavement, by scattering the oil, aids in reducing the slipperiness. A sprinkling of cinders on such a surface also helps greatly.

Ditches and Ditch Slopes Along the Highway

As the wider shoulder serves in an emergency as a turnout, the inner ditch slope, if more flat, may permit a vehicle to go down and come out again without upsetting. If it is too steep it may be the source of many an accident. There seems to be little excuse for making these slopes steeper than 1 on 3.5. In addition, the flatter slopes are more economical to maintain because of lessened erosion and also the reduced cost of mowing the grass and weeds.

In past years deep ditches for drainage have been constructed along highways. One heavily traveled road paralleled by such a ditch for 4 miles has claimed five lives in 5 years. Another stretch (11 miles) shows a property loss to damaged vehicles of $24 000 per year. The public as well as engineers should protest against the practice of a land owner deepening the road ditch in order to drain his farm.

Railroad Crossings

Approaches to Grade Crossings.—Narrow highway approaches to railroad crossings often contribute to accidents. In one such instance four years after the approaches were widened not a single accident has been recorded, although previous to the widening, accidents were common. It is a psychological fact that if a driver's attention is occupied with one thing he cannot so well give it to another. Apparently, this was the only principle involving danger at this crossing. Some advocate a rough road surface at a crossing to slow the traffic and thus increase the safety. It is very likely that all the advantages in slowing down the traffic would be more than outweighed by the detraction of a driver's attention from an approaching train.

Acute Angle of Approach.—At one point in Indiana an acute angle between a main highway and a heavily traveled railroad has been the source of many accidents, due to the wheels of vehicles catching in the groove alongside the rail. This angle (about 15°) is entirely too sharp for safety. It is desirable if not too inconsistent with the alignment, to lay out such crossings at an angle greater than 25 degrees.

Center Posts at Grade Crossings.—At this same crossing center warning posts were erected with a view of reducing accidents. These center posts during the short time of their use, were the source of many times more accidents than they prevented. While such posts may be safe in cities on wide pavements where high speed is not common, they are hardly safe on a wide rural highway where high speed prevails and cannot be so well controlled. Doubtless they would cause many more accidents than the grade crossing itself. While the arrangement might shift the responsibility on the automobile driver instead of permitting the railroad company to take a share, it would not contribute to the safety of such crossings.

Elimination of Railroad Grade Crossings.—Grade crossings on all heavily traveled highways should be eliminated as rapidly as finances permit, either by relocating the roads or by building the necessary structures. Frequently, relocation is the cheaper and at the same time safer method. All structures are small sources of danger in themselves. As formerly built with sharp turns at the approaches, they served horse-drawn traffic fairly well; but with modern traffic such structures are a source of danger only lesser in degree than the grade crossing.

BRIDGE WIDTHS

While narrow roads take their toll, narrow bridges on heavily traveled roads are also a menace. Not only that; due to the frequency with which they are hit and damaged by traffic, in the end they may be more expensive than wider ones. On all important roads, structures of about 20-ft. span or less should carry the full width of the roadbed, that is, pavement plus shoulders. With large structures, because of the cost involved, some sacrifice must be made in safety to fit the treasury. It must also be recognized that any large conspicuous structure, such as a bridge, is more readily observed by a driver and far less apt to be hit than a smaller one, and hence does not need such a wide clearance. On roads designed to carry two lanes of traffic, the proper widths for medium large structures are at least the width of slab needed to carry the traffic plus 8 ft. for primary roads and 4 ft. for secondary roads.

ADVERTISEMENTS AND OTHER OBSTRUCTIONS

Any kind of a billboard or obstruction on the highway, which will detract from the driver's attention, is a contributing factor to accidents. For this reason, as well as for the sake of appearance, all advertisements should be eliminated from the right of way. In most States on the main highways this has been done; in some places, however, they have re-appeared on warning signs. Perhaps all the advantages of the sign are sacrificed by such practice. The habit of painting advertisements on the pavement should be stopped by imposing a severe penalty. Likewise, the method of erecting crosses along the highway where fatalities have occurred is of doubtful value.

Warning Signs

During recent years traffic has increased rapidly, out of proportion to the limited finances for coping with it. Hence, nearly every highway system in America has many dangerous places, some of which cannot be eliminated for years to come. The only recourse is to warn the traveler. Flashlight signals should be installed at practically every grade crossing of an important highway with a high-speed railroad. In Indiana (1927) there are 511 grade railroad crossings on the State Highway System and only about 100 of these are protected by watchmen, gates, or suitable flash signals. Of all fatalities reported on this System, 36% in 1925 and 25% in 1926 occurred at railroad grade crossings (Ohio records show about 30 per cent.). It seems impossible to reduce these fatalities without giving greater protection to the public at such places.

The State should protect the unsuspecting driver against all dangerous places in the highway with ample markers. A standard system for this purpose adopted by the American Association of State Officials seems to work satisfactorily. It should be adopted universally so that the traveler will not be confused by different systems in different States.

Safety campaigns through newspaper publicity have been waged throughout the nation to influence the motorist and thus bring about saner driving. Many people have felt discouraged because of the constantly increasing number of accidents and have concluded that these campaigns were of little value. The writer, on the contrary, believes the most valuable means of increasing safety on highways is to continue such campaigns. Because accidents have not decreased it does not follow that such campaigns have not been beneficial. The greater speeds of travel together with the increased number of vehicles are responsible.

The engineer's duty in the reduction of accidents is a very important one. Although the source of danger preventable by his work is limited, perhaps it can be more easily controlled than any other. The improvements in the automobile bring increases in the speed; the engineer must cope with this problem.

Much can yet be done. Little, however, can be accomplished without study and effort. It is far better to recognize this and put forth every effort to increase safety. The opposite attitude once was taken by a prominent official when he became disgusted with the high-speed traffic and "gave up" with the remark, "Let the damned fools kill themselves". Nothing is so hopeful as making an effort and this is the responsibility and duty to the general public owed by engineers.

DISCUSSION

E. W. James,* M. Am. Soc. C. E.—There can be no possible doubt of the highway engineer's responsibility. The success with which critical danger points have been either wholly eliminated or reduced in hazard by a careful study of specific conditions, as, for instance, in the State of Maryland, establishes beyond question that certain features of highway design and construction are responsible in many cases for the accumulation of accidents, and also that the correction of these conditions may be relied upon to provide considerable remedy.

Mr. Hinkle has not touched upon this detail in a general way and it deserves attention. When Maryland first began collecting accident records, the ordinary wall map with pins to locate the accidents as they occurred, was used. In the course of the report the pins accumulated in significant clusters. The engineers of the State Highway Department made a careful study of each of these dangerous points and without knowing the details of the accidents they were able to find conditions which obviously needed correction. Whether they found all the conditions, both major and minor, which were responsible for the accidents makes no difference. The work done in the way of flattening curves, superelevation, widening, clearing the line of sight, etc., served to reduce the number of accidents at such points significantly. This is obviously the first and simplest method of attacking the accident problem in so far as the reconditioning of existing roads is concerned.

It is obvious that there still will remain scattered accidents, more or less well distributed over the highway system, but to what extent the engineer is responsible for such accidents is more difficult of determination.

The Association of State Highway Officials has recommended that all States adopt the form of accident report used by the National Safety Council. This report, if used, will disclose some of the miscellaneous causes of scattered accidents. A study of the annual record will indicate corrections that may be necessary where clustered accidents do not occur.

One of the important points in the paper appears to be the relativity of dangerous conditions. The combining of sharp curvature and long tangents is an example of this when compared with a road in a mountainous area in which there is considerable sharp curvature with no long connecting tangents. In the first condition the sharp curve is unquestionably more dangerous than in the second condition. Another important point is the avoidance of a combination of bad conditions. As an example, maximum curvature and maximum gradient should not be combined.

Mr. Hinkle mentions a considerable number of design details which have relation to existing hazardous conditions, and attention should be drawn to the two classifications into which these details may be grouped. There is one group susceptible of approximately uniform treatment. In this are width of traffic lane, minimum radius of curvature, maximum gradient, maximum crown, superelevation, easement, widening on curves, and width of shoulders.

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There are other features which are not susceptible to uniform treatment, but among which definite relations may be established. For instance, the succession of curvature, as Mr. Hinkle shows, is likely to affect the safety of the highway where long, easy curves have sharp curves unexpectedly introduced. Sharp curvature on steep grades is more dangerous than in combination with low grades; and sight distance, where highway conditions induce high speed, should be adjusted on both horizontal and vertical curves to favor the traveler.

Just what values shall be assigned to those details which are subject to uniform treatment are still matters of discussion, but there seems to be a marked drift of opinion toward certain values. The Association of State Highway Officials has recommended a width of 10 ft. for each traffic lane as a standard of practice, and this is in harmony with the report of the Hoover Conference on Street and Highway Safety. The speaker suspects that if the comparison made by Mr. Hinkle between the three-lane 30-ft. road out of Indianapolis, Ind., and the two-lane 18-ft. road out of the same city had been made between a 30-ft. road and a 20-ft. road, respectively, the relative number of accidents would not be so markedly against the two-lane road. With the speeds which are common on the public highways to-day, an 18-ft. two-lane road lacks an element of safety which can probably be provided by an additional 2 ft. of width.

The width of shoulder is likewise a detail which can be given general treatment. The Association of State Highway Officials has recommended a minimum width of 8 ft. This will allow ample space for the installation of the necessary markers and caution signs along the shoulders, and will provide space for parking cars, thus removing the inevitable element of danger and interference with traffic which results from parking cars on the surface of the road.

The values to be given maximum gradient will depend to a considerable extent on the general topographic conditions existing, but it is a significant fact that on new roads constructed in the Rocky and Sierra Mountains better gradients have been secured than exists on many of the old established roads in the Appalachian System. This rather effectively disposes of the contention that extreme mountainous conditions demand large concessions in the matter of maximum grades. Perhaps some concession is necessary, but it is probably very slight in any case. It is probable that on main highways a maximum grade of 6% can be secured anywhere in the United States without unduly increasing the cost of construction. It is clear, however, that reduction of grades to this figure on the old established highways in the East cannot be made without substantial relocation and the virtual construction of new roads. The speaker believes that this will eventually happen, and that the main route through Southern Pennsylvania between Philadelphia and Pittsburgh, the old National Road through Cumberland, Md., and others, which are well known and heavily traveled by through traffic, will eventually be replaced by roads of substantially lower gradient. It is significant that the old Northwest Turnpike passing through Winchester, Va., and crossing the Appalachian Mountains to Romney, W. Va., which was laid out by Crozet when he was State Engineer of Virginia, has a maximum gradient of 7%, whereas 10% grades occur on the road in Pennsylvania, and the standard maximum on the National Pike is 9 per cent.

What value shall be given to permissible lengths of different gradients? This is a question which might well be investigated in detail either by the highway engineer or by the designer of motor vehicles. For instance, in the manual* for the erection of caution signs, which has been adopted by practically all States, there is a recommendation to erect a sign advising the traveler to shift gears at the top of a certain grade. The sign should ordinarily be used under the following conditions:

On a 6% grade, more than 2 000 ft. long.
" " 7% " " " 1 000 " "
" " 8% " " " 750 " "
" " 9% " " " 500 " "
" " 11% " " " 400 " "
" " 13% " " " 300 " "
" " 15% " " " 200 " "
" " 16% " , any length.

Obviously, the high gradients appearing in this tabulation have reference to grades on existing roads, because no such grade would ever be designed by a highway engineer to-day; but it is probable that grades of 7 and 8% will continue to be used on secondary roads, and the capacity of the brakes of a motor vehicle to hold the car without themselves burning should be studied to determine the lengths of the several gradients which it is safe to build under existing practice in motor-vehicle construction.

Mr. Hinkle also refers to the widening of pavement at dangerous intersections. The whole subject of highway intersections is up for general review and unquestionably a variety of solutions will have to be adopted, depending on the conditions of traffic found to exist and on the topographic situation in the immediate vicinity. It is an obvious fact that if the traffic on two intersecting highways is up to the capacity of both continuously, or at the daily peak periods, those roads at such times are only 50% efficient. Eliminating highway grade crossings entirely would at once raise this combined efficiency to 100%, but such substantial improvement can only be made where the traffic is very heavy on both routes.

Probably the next most favorable treatment for such conditions is the use of the rotary intersection. This is a detail of highway design which deserves close analysis and study. It is by no means a panacea, but it unquestionably increases the efficiency of an intersection at grade where traffic is heavy, and it is relatively inexpensive. The capacity of such a rotary intersection is probably a function of the radius of the center line of the circular pavement and of the number of routes delivering traffic to the intersection. Theoretically, the width of the circular pavement should be 25% of the sum of the width of all radiating roads. There are not a great many of these rotary intersections in existence, but where they have been used, the success of their operations in handling the large traffic flow certainly warrants further investigation and probably a considerable extension of their use.

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* "Manual and Specifications for the Manufacture, Display and Erection of U. S. Standard Road Markers and Signs," Wash., January, 1927.

Where rotary intersections are not possible, Mr. Hinkle offers a solution which is simple and apparently effective within certain limits. It should be observed, however, in connection with such widening that it does not serve greatly to increase the efficiency of the intersection and is valuable principally as a detail of safe construction. For instance, widened pavement may call attention to a dangerous intersection, but it will only shorten the line of traffic which is forced to stop by the small number of cars that can crowd into the additional width of pavement at the point of stoppage. Such widening might be advantageous at railroad crossings where it would be an indication of a dangerous condition and also permit a slightly greater freedom of manipulation by the driver in case he has to stop suddenly or swerve to one side to prevent accident.

This widening of pavement, as described by Mr. Hinkle, has a rather interesting limitation which once came to the speaker's attention in connection with a design submitted by one of the States on a Federal Aid project. The proposed treatment of an intersection involved a circular connection in each of its four quadrants and a radius of about 800 ft. was used for these curves. The result was a geometrical pattern in which the original intersection had to be left for direct traffic on the two routes and the circular curves, amounting to a total of 360° with all the right of way involved, were introduced merely to take care of that part of the traffic which turned from one intersecting road to the other. Had the entire area within the curves been paved in the form of a widened intersection, it would have been almost as capacious as a small city square.

Finally, the speaker wishes to call attention to the fact that it will be almost impossible to eliminate all dangerous conditions existing on roads now constructed and in service. Whatever engineers may do in new design and in reconditioning, it will be many years before State Highway systems, even on main routes, will be entirely free from objectionable or even dangerous details. Where these exist, the simplest duty of the highway engineer is to inform the public pointedly and timely of the conditions present, and give the driver warning that he is to take care of himself. The standard warning and caution signs adopted by the Association of State Highway Officials are, apparently, very effective in this respect. Public comments favorable to their use have already been received, and requests to extend their introduction have been made in some instances.

A broad study of the conditions producing accidents cannot be made for rural highways to-day. At best, only a few States are obtaining adequate data as to highway accidents. Such studies are likely to be extended, however, and as rapidly as the highway engineer has information, it should be analyzed and its results developed in the details of highway design at every possible point of application.

LOUIS P. BLUM,* M. AM. SOC. C. E.—It frequently happens that the test of the carrying capacity of roads in this country occurs on such holidays as Labor

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* Civ. and Min. Engr., Pittsburgh, Pa.

Day. It would be instructive to have a record of such unusual traffic movements as compared with Sunday or other holidays. It is hoped that if the Ohio Transport Survey has such data they will be made available.

Engineers do not place sufficient emphasis on the necessity of properly, systematically, and regularly posting roads with uniform signs. On railroads, there is no question that the adoption of a uniform system of signals to effect a uniform purpose has produced great results in the matter of saving lives. In the bituminous district of Western Pennsylvania, at the insistence of the insurance companies, the same sign printed in exactly the same manner conveys the same meaning in every mine.

Now, why should not the same uniformity prevail with reference to road signs? There is no more urgent need than this. States, counties, and cities seem to vie with each other in an endeavor to produce highway signs in the greatest variety of style and location. In one of the principal cities of Pennsylvania along the William Penn Highway there are three red lights at three intersecting corners; one of them is on the right side as the driver approaches it; the next one is in the center of the street; and the next one is over on the left side.

Is there any logical reason why, as the driver passes over the State line from Pennsylvania to Ohio, he should find railroad crossings marked in an entirely different manner? Is there any logical reason why a road crossing should be stated in Pennsylvania as being a "road crossing" and in Ohio an "intersection"?

The suggestion has been made that certain highways in the cities should be called "arterial roads."* While this means something to the engineer and to the educated man, it may fail to convey its safety message to the ordinary, uneducated driver seeking to thread his way through the maze of traffic of a strange city.

Mr. Hinkle has well said that "if a driver's attention is occupied with one thing he cannot so well give it to another", and it seems to the speaker that highway safety demands the adoption of an absolutely uniform system of signals throughout the United States—uniform as to shape and color.

C. N. HAGGART,† ASSOC. M. AM. SOC. C. E.—In driving over various highways, the speaker has noticed that drivers occasionally take the inside of the curve in making a left-hand turn. Would it not be good engineering and designing for safety, to increase the superelevation on the outside of the curve and thus encourage drivers to keep to the right side of the road? In some places, the outside of the curve looks very dangerous, particularly for making a left turn on a down grade, with a steep declivity to the right. Even a cautious driver may be tempted to drive to the inside of the curve under such conditions.

It is the speaker's understanding that the Bronx Parkway Commission of New York has a standard of design similar to this; the only difference being that the superelevation between the inside and outside lanes of traffic is decreased. As far as is known, this method of superelevation has proved very satisfactory on the Parkway. The idea might be well worth investigating for the sake of safety on the highways.

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* *American City*, December, 1927, p. 761.

† Structural Engr., Pittsburgh, Pa.

H. J. Kirk,[*] Esq.—In connection with superelevations, it is well to keep in mind that conditions vary on a rural highway as compared with cities and villages. The superelevation is a function of speed in the vehicle and it must be remembered that on rural highways there is a mixed traffic and that it is possible to make the superelevation so high that trucks are inconvenienced by it, particularly in winter when the highway is covered with ice. This complaint has been made on some highways that carry heavy truck traffic. The superelevation is fine for motor cars, but it seriously inconveniences some of the heavy trucks because they cannot maintain the speed necessary for that superelevation.

There have recently appeared some advocates of a so-called rotary intersection. Theoretically, this appears to be very fine, but it is not practicable where there is any amount of pedestrian traffic. It is noteworthy that where these intersections have been tried in Columbus, Ohio, they have been abolished.

W. W. Crosby,[†] M. Am. Soc. C. E. (by letter).—It seems to the writer that Mr. Hinkle has performed a signal service by analyzing the situation and pointing out so forcefully the duties of the engineer.

It may be necessary, in order to bring home to Americans the realities of a situation, to personalize the matter of safety and to emphasize or dilate on the financial costs of accidents to individuals as has so often and so generally been broadcast in recent years. In countries where highway problems are older and often better comprehended, the safety of the individual is subordinated to the safety and continuity of the traffic as a whole. Thus, the problems may be approached with less hysteria and with calmer judgments exactly as Mr. Hinkle has tried so well to meet them.

Under the head of "Widening on Curves", the remarks made seem logical, although the writer cannot agree to any deduction that "spiraling" should not be used on "highway curves".[‡]

Although Mr. Hinkle did not list "Advertisements and Signs" among the causes of accidents for which the engineer may properly be held directly or indirectly responsible (according to his actual authority), they are mentioned pertinently near the end of the paper.

The writer thinks there are, on the whole, and certainly often, too many signs along the highway. "Wolf! Wolf!" is cried too often with the traditional effect. So-called "informative signs" obscure the vision, distract the attention, and impair the caution of the drivers unprofitably. These all certainly fall within the scope of the engineer.

When it comes to advertising signs, of the same or worse consequence, words fail to express the situation adequately. The writer firmly believes that no advertising signs should be permitted within 200 ft. of the highway except for business done on the premises, and then only under the approval (as to location, size, and type or form) of the highway authorities. In built-up sections, of course, the rule would have to be modified.

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[*] Director of Highways, State Dept. of Highways, Columbus, Ohio.

[†] Cons. Engr., Coronado, Calif.

[‡] "Road Corners and Junctions," by Rees Jeffreys, Chairman, Roads Improvement Assoc. of Great Britain, *Municipal Journal and Public Works Engineer* (Lond.), January 6, 1928.

The writer confesses to a sympathy with the "prominent official", quoted in the author's last paragraph, when the eccentricities of a few drivers, or the "peaks" of average driving, run too far outside a reasonable "load curve" for safe use of well-designed highways. The engineer may well be ambitious to offer remedies for public benefit, but he may equally well refrain from going too far in attempting to insure the survival of the unfit. Some persons, in the exercise of their undeniable rights, may prefer to "die with their boots on" a throttle, and the engineer, like others, may do best by "sticking to his last".

VICTOR A. ENDERSBY,* M. AM. SOC. C. E. (by letter).—This subject is one of increasingly vital importance to all highway engineers. Safety is now as important on highways as it ever was on railroads. It is continually more difficult to attain, and there is a marked increase in the public tendency to blame mishaps on the engineer. More than that, members of the public are becoming so familiar with the possibilities of engineering, that they can criticize lapses with hitherto unknown accuracy and lack of diffidence.

As Mr. Hinkle remarks, no exact line can be drawn. One can place a finger on a given set of circumstances and say "reckless driving", and upon another and say "bad engineering", with considerable certainty. The line lies at an indefinite location between, so that there will probably never be a legal definition of it.

The author, however, states that "about 30% of all accidents recorded in Indiana in 1926 were due to highway conditions". How can the figure be so definite? To say that any accident is due to "highway conditions" without qualifications is to say that no driver could pass that point without an accident. Driver responsibility cannot be eliminated from any accident; engineering responsibility can be reduced to an infinitesimal. Engineers and drivers both need education, but the handling of the latter is the more difficult.

Alignment and Grades on Bridges.—Bad grades or lines are more definitely localized and noticeable at bridges than elsewhere. The effect is made more acute by the fact that most bridges are narrower than the roadbed.

The State of California has inherited a vast number of bridges built on the ancient principle of crossing the stream (or railroad track) at right angles, and torturing the line to conform to it. This is a twin to the idea that, since bridges are few and far between, it is no hardship for traffic to slow down for an occasional narrow "gut"; hence, there are numerous bridges only 18 or 20 ft. wide, on main lines. The first fallacy has resulted in numbers of bridges on atrocious alignment, lying in wait to catch the driver lulled into a false sense of security by a long stretch of wide, straight, modern highway. The second forces the constant meeting and passing of machines on bridges too narrow for safety, and, in some cases, chokes the traffic to an actual standstill.

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The bridge-building program of California is years behind the highway development, for reasons not within the scope of this discussion; and although the entire State Administration is anxious to remove this condition as fast as humanly possible, it will be years before it is wholly remedied. The contrast of some of the bridges with the highways leading to them has to be seen to be appreciated.

One definite step toward improvement can be taken by nearly any organization responsible for bridges; a step which at least will prevent the condition from becoming worse; that is, a policy of no compromise with bad alignment and narrow widths. Bridges should be subordinated to roads, not the reverse. If the funds are not available with which to build a bridge properly, the building should be deferred until they are, barring actually desperate conditions. This is the present working policy of California and it is justified both morally and economically. If a bridge is built which will merely better the traffic conditions slightly, it will have to be rebuilt sooner or later at great cost, meantime remaining a public danger. If the situation is serious, and funds are limited, it is better to build a structure of cheap materials on good line and grade, than a permanent structure which will have to be altered or superseded. Experience in California has shown that the life of a concrete bridge, if it is a little too narrow or on line a little too crooked, is so short that a wooden bridge on proper location will outlive it.

It is astonishing how many existing bridge alignments could have been bettered with little or no additional cost, by an intelligent study of topography and competent selection of design.

There are four common faults: First, the conventional practice of going around hills rather than through or over them (without due investigation of comparative costs); second, locating engineers allowing themselves to be bluffed by topography which looks formidable; third, unfamiliarity with the relative economy of types of construction; and, fourth, insufficient analysis of relative costs of piers and superstructure.

Again and again bridges are built at right angles to save length, where a longer bridge of another type could have been built on good alignment for the same money. All this would seem merely engineering platitudes, except for the fact that they are often neglected in unexpected quarters.

Widths of Bridges.—Present California practice (1928) is to use multiples of traffic lanes 10 ft. wide, plus 2 ft. on each side, the additional 2 ft. being omitted where there is a sidewalk. The widths are thus 24 ft., 34 ft., 44 ft., etc., except where special conditions require other widths. Mr. Hinkle's 8 ft. as against California's 4 ft. is, of course, an improvement, and is probably desirable where there are less than three lanes. Where there are three lanes, or more, it is not so important.

Bridges less than 34 ft. wide are becoming obsolete on California State Highways. One bridge is being constructed 100 ft. wide, and another, several hundred feet long, is being widened to more than 60 ft. Frequently, bridges 44 ft. and 34 ft. wide are built on 20-ft. pavements, a reversal of former practice. The apprehension as to such bridges is not that they will be too wide, but that they will be too narrow within a few years; but there is a limit to which the public will permit the expenditure for wide bridges on narrow pavements.

Railroad Crossings.—There is no such thing as a good grade crossing. In California all railroad crossings are to be eliminated where possible and are tolerated only under duress elsewhere. Consideration of their relative danger enters the State policy only as a matter of deciding which shall be eliminated first with the use of funds that will always be insufficient, however large in absolute figures.

This is a difficult decision, although one which constantly has to be made. Charles E. Andrew, M. Am. Soc. C. E., Bridge Engineer for the State, has suggested that a formula could be developed giving reasonable mathematical values to the various danger factors, and that crossings could thus be set up for elimination by a mathematically rational method. The problem, however, is so involved that no one has made much progress with it. Meanwhile, California has so many bad crossings that no money is in danger of being spent on needless eliminations for years to come.

Superelevations on Roads.—All curves on California highways of less than 1 500 ft. radius are now built with superelevations. Mr. Hinkle suggests that superelevations steep enough to cause sliding to the inside under ice conditions, could be taken care of by the use of a flat inner shoulder. The writer regards that suggestion as dangerous.

If there is real danger from this source, the superelevation should be reduced. Except where the bank is very high or precipitous, it is not as dangerous to skid off the outside as to encounter another machine on the "blind" side while out of control.

Reduction of Curvature.—The suggestion that the sharpness of a curve be limited with relation to that of the last one preceding it is a valuable one which will be endorsed by most experienced engineer drivers. The writer is familiar with situations where this would have saved much loss of life and property. It would seem that where a long stretch of easy alignment breaks into country where sharp curves are inevitable, considerable expense would be justified in the gradual reduction of radius for the first mile or so. Has this ever been tried?

Widening.—Some widening, to take care of the greater difficulty of steering and the traffic situations peculiar to curves, seems desirable. Beyond a certain point, however, widening on the inside is merely equivalent to a reduction of the curvature, owing to the tendency of machines to crowd to the inside. Mr. Hinkle's figure of 0.3 times the sharpness of the curve, in degrees, giving 1.5 ft. for a 5° curve, and 3.0 ft. for a 10° curve, seems more reasonable for the latter than for the former. Driving around a 5° curve is little more difficult than driving on a tangent. How was this formula derived?

There is, for standard machines, a certain critical curvature for each speed; on curves sharper than this, a widening of even 2 or 3 ft. on the inside produces an astonishing difference in safety of driving. Sharp curves on California highways are being progressively reduced by cutting back and adding oiled gravel shoulders in the course of maintenance work. It seems desirable to make studies to determine the best minimum curvature for prevailing conditions—with a due allowance for future "improvements" in the speed of vehicles.

Considerable safety can be added by cutting back the slope on curves, leaving a berm about 4 ft. high over which the driver can see coming machines; the remaining bank keeps him on the original alignment.

Rough spots are more serious on curves than on tangents. A bump which would not matter in the latter case, will break tire traction and cause a skid in the former, owing to centrifugal force.

Crowning.—There is no longer a reason for crowns of more than 2 or 3 in. in 20 ft., except for dirt roads. California State highway pavements are crowned slightly to prevent water standing on the surface. Parabolic crowns were abandoned some time ago, the cross-section now being two straight lines. This makes for safe driving and easy steering. Some counties build pavement with no crown, but with a tilt of ½ in. to 1 in. for drainage. From some points of view this seems even more desirable, because drainage needs to be handled on only one side of the road at a time. There seems little logical reason for crowning on superelevations.

Pavement Edges.—It is a practical impossibility to maintain perfect shoulders on all concrete pavements; ruts form at the side. When machines run off the pavement, or are forced off, they cannot be brought back without a severe wrench, and if they are going at any great rate of speed, they may come back so nearly out of control as to weave across the road. This is a prolific source of accidents. Beveling the edge of the pavement sufficiently to permit the tire to climb it without a jolt might be advisable; it would certainly postpone raveling.

Grades and Curves.—The writer agrees with what is said about grades and curves. Mr. Hinkle, however, does not mention the effect of grades in slowing down trucks and thus congesting traffic. It would be well if the grades which cause the average truck to shift gear could be determined, and avoided on critical points, such as blind curves. Sometimes a long stretch of good grade is interrupted by a short piece of steep grade where climbing trucks can be depended upon to "bottleneck" the whole stretch. In general, alignment should never be sacrificed to grade from the safety point of view. Economically, the consideration tends to the other extreme, especially for slow heavy traffic.

Traffic Separation.—The writer believes that the day is in sight when freight traffic will be increasingly separated from light car traffic, on roads of its own. This will bring about two distinct standards of highway engineering, not only as to alignment, but as to pavement and bridge construction. Observation of traffic conditions in Southern California leads one to believe it impossible that public patience will tolerate the present awkward mixing for many more years. The attempt to handle hundreds of freight vehicles, including trucks and trailers with a combined length up to 50 ft. and a weight of 30 tons, over the same two-track road with large-volume light-car traffic, is simply foolish.

It seems inevitable that major highways of the future will be four-line; there will be center lines for through traffic with no upper speed limit, but with minimum speed at 30 or 40 miles, and side tracks for local and slow traffic.

Width and Safety.—Without doubt width contributes to safety. The cause of most accidents is the inability of the driver to maneuver, or to maneuver in time. On a wide road he not only has room to handle the machine and enough space to see what is transpiring ahead, but machines also tend to keep away from one another.

Rights of Way.—The standard width was 60 ft. when the original system of California State highways was laid out in 1912. The people of the State are now paying dearly for this, because of the necessity of getting 80 and 100-ft. rights of way through, not merely rich agricultural land, but miles of country which, although unincorporated, is urban in character. The minimum legal width is now 80 ft.

It is difficult to understand why narrow rights of way should ever be tolerated. In thickly populated districts the time to get wide rights of way is as soon as possible before values rise; in the country, the additional cost is low—and no one knows how long it will remain so.

Advertisements.—Signs are forbidden within the rights of way of California State highways. Unfortunately, however, much pavement is built practically up to the right-of-way line in just the locations where signs are thickest; that is, in the semi-urban districts. There is need for legislation forcing signs to be placed a definite distance back from the right of way, and also to prevent them from being placed in fields so as to cut off the view of the road ahead, or of tracks in case of crossings.

The tendency of "hot dog" stands, service stations, etc., to be crowded as close as possible to the right of way along country roads, thereby converting these roads for all practical purposes into city streets, are evils of a similar nature. Not more than one-half these obstructions pay their way, and it would be to the good of humanity if some means could be found, not only to force set-backs, but to eliminate a large percentage of the buildings altogether.

Pedestrian Traffic.—California highways are lined with pilgrims of all descriptions, young and old, who walk on the pavement, signal motorists for rides, and often shout at them. They force the driver to swerve into the center of the road, frequently distract his attention from his driving, and check and endanger traffic by causing machines to stop to pick them up. Legislation is proposed to make it a misdemeanor to walk anywhere except on the left-hand side of the road. This would help in so far as it is enforced.

Warning Signs.—The writer has not much faith in warning signs as now used. He is rather inclined to think that drivers who are careful enough to notice ordinary signs will take care of themselves anyway, always excepting cases where the road presents dangers not normally to be expected, such as a sudden sharp turn at a bridge, etc. On bridges under construction, detour signs the size of a house, well lighted, are constantly being driven into and knocked into kindling wood. One driver passed three red-and-white, "Danger—Bridge Out" signs, 4 by 6 ft., spaced several hundred feet apart; he wrecked three out of five red lanterns, all burning, and smashed an 8 by 10-ft. detour sign with 5-in. red reflectors. When he was pulled out of the wreckage, he wanted to know why the "blank-blanks didn't have some lights out". He was not drunk.

No means of warning the public should be neglected; but warning signs are by no means substitutes for safety engineering. Great care should be used in placing signs. On California highways, there are too many warnings of curves ahead, placed without much regard to the relative danger of the various situations. The same sign is used to notify of curves which can be driven at 40 miles, and those which are death-traps at 25 miles. Drivers become accustomed to disregarding signs and using their own judgment. Warning signs should be used sparingly, only where real danger exists, and then should be made as prominent and emphatic as possible.

Money spent on signs, carefully considered and designed, could be saved by the elimination of a great number of ineffective ones. There is a crying need for extended psychological study in this field.

Education.—Education can do much, but it cannot alter eyesight, machine-sense, or speed of reaction. It can partly eliminate mental carelessness, but cannot level the discrepancy between machine power and flesh and blood. The best driver can hardly get the brakes on in less than ½ sec. after sighting an unexpected danger. This is nearly 30 ft. at 40 miles per hour, and a highly fatal distance under many circumstances, to say nothing of the time required to stop after that. Every one curses the habitual fools; but hardly any one can say that he does not drop to their level upon occasion, through weariness, temporary nerve depletion, abstraction, or sheer ennui. The automobile has created a schedule of living which necessitates fatalities as absolutely as does the staking of flesh against machinery in war. The engineer cannot hope to catch up with the development of the automobile, but he can save lives, by pushing his methods of construction everywhere possible over the narrow margin between safety and destruction for the average driver.

G. C. Dillman,* M. Am. Soc. C. E. (by letter).—A review of the paper indicates that Mr. Hinkle has covered this subject in a very complete and able manner. The writer agrees that the highway engineer is responsible for highway safety to the extent of the design, construction, and maintenance of highways, and that it is the highway engineer's duty to "keep abreast" with all modern developments pointing to increased traffic study. The limits set by Mr. Hinkle relative to sight distance, degree of curve, widening and super-elevation of curve, allowable grades, and his treatment of the railroad-crossing problem, all represent sound engineering practice.

In connection with statements on widths of pavement, the writer believes that 20 ft. is the best standard minimum width for main highways, and that when traffic demands an increased width, it should be 40 ft. rather than to 30 ft. In other words, the road surface should be such as to accommodate either two or four lanes of traffic, and the three-lane surface should be avoided if possible.

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* State Highway Commr., State Highway Dept., Lansing, Mich.

In connection with shoulder widths the writer would fix 8 ft. as the minimum width for main highways as being none too wide to accommodate the parking of a truck for repairs. Shoulders should be maintained in such condition as to permit traffic to turn out on them safely in emergencies, and to park on them for repairs.

The writer believes that 100 ft. should be the minimum width of right of way for all main highways, and that in cases of very heavy traffic on roads adjacent to large cities the right of way should be 150 to 200 ft. wide. While this might appear to be extravagant, in some cases, if the adjacent property is very valuable or highly developed, it is probable that this property will increase in value and that money will be saved ultimately by purchasing the necessary land now. Ample right of way is one of the chief requisites for future safety developments.

Mr. Hinkle's comments on the necessity of avoiding the construction of highway surfaces of a slippery type, or the use of maintenance methods which promote a slippery surface, are very heartily endorsed. The practice of sanding icy pavements, particularly on grades and curves, is suggested by the writer as good maintenance practice.

The widening of approaches to railroad grade crossings is very desirable, and such improvements should also contemplate the increase of a clear view at such crossings. An attempt is made in Michigan to create a clear view on highways between the driver of a car 300 ft. from the crossing on the highway and the train 600 ft. from the crossing on the railroad. This triangle, if possible, should be kept clear of buildings, trees, brush, or farm crops other than low growing varieties. Condemnation of such property is believed to be warranted if it cannot be purchased otherwise. It is thought desirable to have the angle between a railroad and highway not less than 45°, and preferably 60°, in order to give the motorist a better view of approaching trains.

Narrow bridges are a distinct menace to traffic, and every effort should be made to have all culverts and small-span bridges constructed to the full width of the highway grade. Mr. Hinkle's point that bridges of longer span are more conspicuous and usually promote more cautious driving than smaller bridges, is well taken. The maintenance of bridge floors and railings is an important item in safety to traffic, and it is highly desirable that any bridges which are not capable of bearing full loadings should be posted, with the safe load limit indicated. Thorough inspection of all bridges twice a year should be standard practice.

The construction of "by-pass" routes to carry through traffic around congested cities rather than through business sections is a feature of highway work which promotes not only safety, but convenience of traffic. In suburban areas, where there is considerable pedestrian traffic on highways, it is highly desirable to provide footpaths back of the ditch lines. All highway bridges in such areas should include sidewalks. The construction of pedestrian tunnels underneath heavily traveled roads is an important safety measure, particularly in the vicinity of school houses.

Ample guard-rail protection on high fills is very desirable. The old type of wooden guard rail is a distinct hazard in modern traffic, as many fatal accidents have occurred when cars, striking wooden rails, have had sections of rail pass through the radiator or wind-shield. Cable guard rail is considered preferable to wooden rail, not only from the standpoint of safety, but because it does not drift snow on the roadway like the wooden rail.

Careful attention should be given to the maintenance of gravel roads with reference to the elimination of the two traffic hazards, loose gravel and dust. The use of some dust palliative, such as calcium chloride or light oil, is a prime requisite under modern traffic conditions. Sufficient binder material should be placed on gravel roads to bind the loose gravel, or, if this is impracticable, the excess loose gravel should be carried on the edge of the roadway during the heavy traffic season and brought back into the roadway in the fall where it will tend to consolidate into the road under fall rains.

The careful selection and maintenance of detours with a view to minimizing traffic hazards is receiving increased attention from highway engineers every year. The winter maintenance of highways should provide sufficient patrol of the surface to avoid dangerous ruts in the snow.

A class of highway traffic which has developed rapidly is the moving of large equipment, such as steam shovels, concrete mixers, etc., on large trailers. Movements of this kind should be allowed only under special written permit from the highway authorities in charge, in order to afford proper protection against destruction of roads and bridges, and to safeguard other traffic while it is passing this oversized load. Movements of this kind should be confined to daylight hours, as a vehicle of this size is a distinct menace to other traffic at night.

The engineer should make an intelligent study of the causes of traffic accidents and develop a systematic practice for increasing the safety of the roads. This may be brought about through traffic surveys and research, more adequate road design and construction, better maintenance, pavement markings, and the installation of signs and signal devices. Standardization is a prerequisite to safe traffic conditions, and the engineer is in a position to exert much influence in the standardization of engineering features and the development of traffic regulations which should be uniform in character.

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Paper No. 1738

THE SEWAGE DISPOSAL WORKS OF DECATUR, ILLINOIS*

By Samuel A. Greeley,† M. Am. Soc. C. E.,
and William D. Hatfield,‡ Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. George W. Fuller, John F. Skinner, and Samuel A. Greeley and William D. Hatfield

Synopsis

Since 1917 the Sanitary District of Decatur, Ill., has completed intercepting sewers, pumping stations, and treatment works for sewage disposal at a total cost for construction projects of about $1 700 000. The treatment of the sewage has been complicated by the waste from a large starch works which has increased the human population of about 50 000 to a total equivalent population of 300 000, more or less, during periods of heavy corn grinding at the starch works. The peculiar character of the sewage has required the operation of two testing stations prior to the design of the sewage treatment works. A record of the steps taken and of the works built in connection with the development of this project is described in this paper. The writers' connections with the work were as Consulting Engineer and Superintendent, respectively, for the Decatur Sanitary District.

General Statement

The Problem and the Work Done.—For a year or so prior to 1912, the pollution of the Sangamon River below Decatur, Ill., was sufficiently marked to arouse complaint and the situation was brought before the Rivers and Lakes

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* Presented at the meeting of the Sanitary Engineering Division, Columbus, Ohio, October 13, 1927.

† (Pearse, Greeley & Hansen), Chicago, Ill.

‡ Supt., Sewage Disposal Plant, Decatur, Ill.

Commission, which had jurisdiction over stream pollution in Illinois at that time. The city had a population of about 35 000 and included among its industries a starch works with a daily grind of 5 000 to 10 000 bushels of corn. The sewage from the starch works had a relatively high population equivalent and it was soon recognized that special study would be required to determine the most suitable method of treatment. All the existing sewers were combined, with four main outlets into the river below a low channel dam at the water-works (Fig. 1). For several months nearly every year, almost the entire flow of the river was diverted to the water supply and discharged again into the river as sewage. Thus, an increased water supply through storage on the river was indicated.

Projects for intercepting sewers and sewage treatment were developed in 1915 with an estimated cost of $730 000. This relatively high sum made financing difficult, so that ways and means were carefully studied by city officials and citizens' committees. As a result, the Sanitary District Act of 1917* was prepared and passed by the Legislature. All the construction work for sewage disposal has been done under this Act, amounting all told to nearly $1 700 000. The city has since grown to about 50 000 inhabitants and the grind of the starch works to as much as 45 000 bushels in 24 hours.

The work done may be briefly summarized as follows:

> (*a*) The necessary field surveys and office studies for the determination of intercepting sewer and sewage treatment projects.
>
> (*b*) The operation of two testing stations. One comprised an Imhoff tank and sprinkling filter to determine the behavior of the mixed domestic and industrial sewage. This station was operated in 1914 and again in 1917. The other comprised a tank with a Simplex surface aerator for studying the partial or pre-aeration of the Imhoff tank effluent before applying it to sprinkling filters at the main sewage treatment plant. This work was done during 1925 and 1926.
>
> (*c*) The construction of an intercepting sewer 18 317 ft. long and a sewage treatment plant comprising a grit chamber, six Imhoff tanks, sludge-drying beds, six pre-aeration tanks, two settling tanks for aerated Imhoff tank effluent, 3 acres of sprinkling filters, one final settling tank, and appurtenances. There are two small pumping stations for low-level districts. Actual construction work was started in 1919 and completed in 1927.

In addition, the District has done a certain amount of cleaning work along the river; has acquired sites and right of way; has investigated coal mine subsidence; has fought numerous legal battles; and has carried on a vast amount of administrative work. Under conditions found at Decatur, the completed sewage treatment plant has a capacity for a population equivalent of 150 000.

The City and the Sanitary District of Decatur.—The City of Decatur is on the Sangamon River about 50 miles by river above Springfield, Ill. The drainage area of the river above Decatur is about 862 sq. miles. The city has an area of 4 885 and the District of 21 120 acres. The trend of population is given by the following figures, the design having been based on an ultimate actual population of 120 000:

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* *Transactions,* Am. Soc. C. E., Vol. 91 (December, 1927), p. 441.

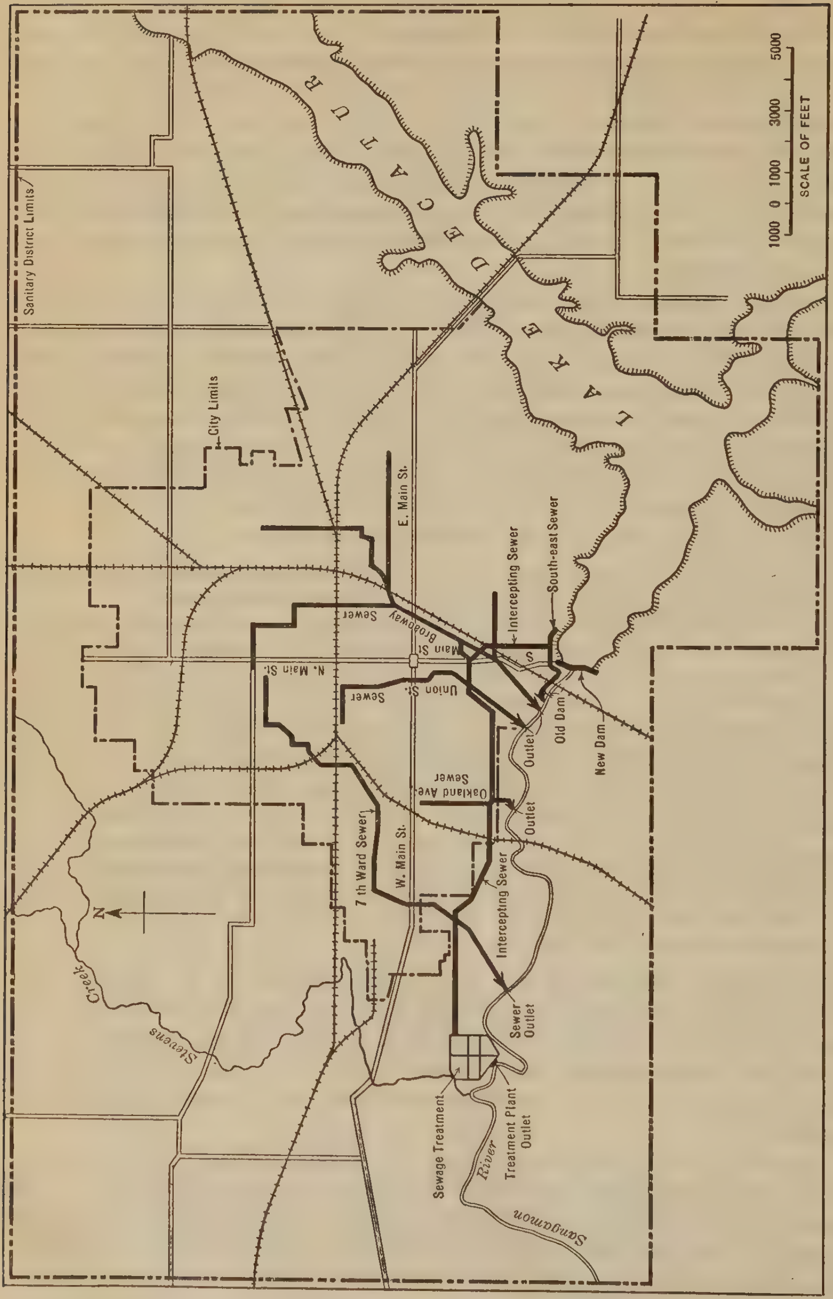
Sanitary District Limits
City Limits
LAKE DECATUR
SCALE OF FEET
1000 0 1000 3000 5000
E. Main St.
Intercepting Sewer
South-east Sewer
Broadway Sewer
Main St
S
N. Main St.
Union St. Sewer
Old Dam
New Dam
Outlet
Oakland Ave. Sewer
Outlet
7 th Ward Sewer
W. Main St.
Intercepting Sewer
N
Stevens Creek
Sewer Outlet
Sewage Treatment
Treatment Plant Outlet
Sangamon River

FIG. 1.—SANITARY SEWER LAYOUT, DECATUR, ILLINOIS.

Year.	Population.*	Year.	Population.*
1890	16 841	1930	57 500
1900	20 754	1940	71 000
1910	31 140	1950	84 000
1920	43 818	1960	99 000
1927	53 400		

The city is built mostly to the north of the river on ground 50 to 100 ft. above mean river level. This high ground is cut by several relatively deep watercourses extending northerly through the city, one or two of which have been drained and filled. The river valley is upward of ¼ mile wide and, at present, the sewers reach the margin of the valley 20 ft. or more above mean river level. For the most part, ample drainage is available.

TABLE 1.—METEOROLOGICAL DATA.

Month.	Average temperature, in degrees Fahrenheit.	Prevailing winds.
January	26.5	NW.
February	27.3	NW.
March	40.9	SW.
April	52.0	SW.
May	63.3	SW.
June	72.0	SW.
July	76.3	SW.
August	74.8	SW.
September	67.8	SW.
October	55.8	SW.
November	42.2	NW.
December	30.3	NW.

The city has a large variety of industries, including a starch works, a structural steel company, railway shops, and many others. It is served by the Wabash, Illinois Central, Pennsylvania, Chicago and Indiana Western Railroads, and the Illinois Traction System.

Meteorological Data.—Decatur is located in the corn belt of Central Illinois. The average temperature and the prevailing winds are given in Table 1.

TABLE 2.—MEAN MONTHLY RAINFALL.

Month.	Rainfall, in inches.	Month.	Rainfall, in inches.
January	2.46	July	3.13
February	1.99	August	3.18
March	3.17	September	3.47
April	3.53	October	2.26
May	4.03	November	2.31
June	3.59	December	2.13

The weather retards construction work seriously during about three months of the year. Sludge drying on open beds can be done during about eight months. Sprinkling filters can be operated throughout the year, with reduced nitrates in the effluent through the winter months.

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* The population figures for 1890, 1900, 1910, and 1920, were taken from the United States Census reports; those from 1927 are estimated.

The mean annual rainfall is 35.05 in., with a mean monthly rainfall as shown in Table 2.

During wet seasons, the ground-water stands high and there is a marked infiltration into the sewers which increases the volume and lowers the temperature of the sewage. Unusually heavy rains occurred in the late summer and fall of 1926, as follows:

Month.	Rainfall, in inches.
July	2.23
August	7.47
September	16.56
October	6.18
November	3.09

The Sangamon River.—The Sangamon River, with a drainage area of 862 sq. miles above Decatur, flows through a relatively wide valley of cultivated bottom-lands and wooded areas which provide a large flood-water storage. Floods, therefore, are of relatively low intensity and long duration. There are United States and Illinois Geological Survey gauging stations on the river at Monticello, 27 miles above Decatur, and at Riverton, 42.5 miles below Decatur, both being approximate distances by river. The maximum flood of record was at the rate of about 25 cu. ft. per sec. per sq. mile. The mean monthly flow at Decatur, estimated from the gaugings, is as follows:

Month.	Mean monthly flow, in cubic feet per second.	Month.	Mean monthly flow, in cubic feet per second.
January	577	July	354
February	1 118	August	277
March	916	September	200
April	1 047	October	233
May	1 133	November	202
June	466	December	142

Two periods of very low flow occurred as listed in Table 3.

TABLE 3.—Record of Low Flow.

Months.	Mean Monthly Flow, in Cubic Feet per Second.	
	1914.	1920.
July	12.0	67.5
August	5.6	15.8
September	10.4	16.9
October	6.9	12.4
November	8.6	19.6
December	13.0	27.6
January	12.0	82.8

Historical Statement

Situation in 1913.—The condition of the Sangamon River resulting from the domestic and industrial sewage of Decatur, became sufficiently offensive in the years prior to 1913, so that complaint was made and the assistance of the

Illinois State Water Survey was secured in order to compile a complete report. During 1912 and 1913, therefore, this organization made an extensive sanitary survey of the Sangamon River with special reference to pollution by the Decatur sewages. Its report included complete chemical and bacteriological analyses of the Sangamon River for a distance of about 20 miles below Decatur and recommended the installation of settling tanks and sprinkling filters.

Rivers and Lakes Commission Order, 1914.—Following the Water Survey report, further hearings were held by the Rivers and Lakes Commission and the Consulting Engineer for the City, Langdon Pearse, M. Am. Soc. C. E., made a report indicating the difficulties of the problem, due in part to the industrial sewage, and outlining the investigations necessary to a proper solution. As a result, the Commission ordered the pollution of the river to be removed by January 1, 1917, thereby allowing (as it thought) sufficient time for a thorough and careful study of the problem.

Pearse and Greeley Report of 1915.—On June 4, 1914, the City instructed Messrs. Pearse and Greeley, as Consulting Engineers, to undertake the necessary investigation, and their report was submitted on April 23, 1915. One of the major problems calling for special study was the effect on sewage treatment processes of the industrial sewages found in Decatur. Consequently, funds were appropriated for a testing station which was operated during the last half of 1914. The station was located near the mouth of the Broadway Sewer into which the wastes of the starch works, gas plant, and packing-house were discharged, in addition to the domestic sewage of 13 700 people. Unfortunately, the outbreak of the World War caused the starch works to shut down, so that tests were limited chiefly to the domestic sewage, as the other industrial wastes were very small in quantity. The station included a grit chamber, an Imhoff tank, a sprinkling filter, and a secondary settling tank. With a settling period of about 1.78 hours and the sprinkling filter dosed at an average net yield of 1 330 000 gal. per day per acre, 6 ft. 4 in. deep, the results of operation are shown in Table 4.

TABLE 4.—AVERAGE TESTING STATION RESULTS, OCTOBER 24 TO DECEMBER 2, 1914.

Substance.	Amount in influent, parts per million.	Amount in effluent, parts per million.	Percentage of reduction.	Percentage of increase.
Imhoff Tank:				
Suspended matter	221	108	52	......
Bio-chemical oxygen demand	171	167	2.3	......
Organic nitrogen	12.0	9.9	17.5	......
Free ammonia	14.6	15.6	......	6.8
Nitrites	0.12	0.12	......	0.0
Nitrates	0.47	0.47	......	0.0
Oxygen consumed	53	49	7.6	......
Sprinkling Filter:				
Suspended matter	108	40	63	......
Organic nitrogen	9.9	1.8	82	......
Free ammonia	15.6	3.8	76	......
Nitrites	0.12	0.46	......	284
Nitrates	0.47	10.0	......	2 025
Oxygen consumed	49.	18.	62	......
Secondary Settling Tank:				
Suspended matter	34	23	32	......

The report developed projects for intercepting sewers and sewage treatment works, recommending a high level sewer and a treatment plant comprising clarification works and sprinkling filters. It also included a careful discussion, with estimates of cost, of the activated sludge process in accordance with data then available.

Test of Lime-Electrolytic Process, 1915.—In 1915, the Electrolytic Sanitation Company made an offer to the City to build a small plant and to test its process in Decatur, with certain guaranties of its efficiency. The offer was accepted, the plant built, and on July 30, 1916, a 28-day test was begun. At the conclusion of the official test, the plant was given over for the City to operate.

The conclusions drawn by the City's representative may be summarized as follows: First, the process did not furnish a stable effluent except when excessive quantities of lime were used; second, the operating and upkeep costs were prohibitive; third, the method of sludge disposal was unsatisfactory; fourth, there was ample reason for differences of opinion as to whether the electrolytic action contributed materially to the efficiency of lime precipitation; and, fifth, the iron electrodes were subject to rapid corrosion—a new bank of electrodes was completely dissolved during two months' operation. On the basis of these findings, the electrolytic system was dismissed.*

Notes on Activated Sludge, 1916.—In April, 1916, the Consulting Engineers worked up a project for an activated sludge plant located close to the outlet of the largest existing sewer. On the basis of securing power at 1.25 cents per kw-hr., the following total annual costs were shown:

1914 Report:	
Sprinkling filters	$58 572
Activated sludge, 2 cu. ft. air	90 188
1916 Notes:	
Activated sludge, 2 cu. ft. air	$68 360

The lower cost in 1916 as compared with the 1914 estimates for activated sludge is partly due to a much reduced intercepting sewer cost and more favorable assumptions for excess sludge disposal.

At the request of the Decatur Association of Commerce, John W. Alvord, M. Am. Soc. C. E., was engaged to review the work done on sewage disposal and to report on the water supply. This report was submitted in May, 1917. It characterized the recommendation of the Consulting Engineers as "an obvious and conservative method of dealing with sewage"; and then suggested further investigation of two alternative projects—one comprising settling tanks and sand filters with a smaller intercepting sewer; and the other, settling tanks at each sewer outlet discharging into a so-called dilution pool formed by a low dam across the river.

From January to August, 1917, H. R. Lee, City Chemist, operated the testing station built in 1913. He reported to the City as follows:

> (*a*) The Imhoff tank and sprinkling filter system will produce a stable effluent.

---

* *Engineering Record,* November 11, 1916.

(*b*) The maximum rate at which a 90% stability could be maintained was found to be 1 000 000 gal. per acre per day. Whether this rate is sufficiently high to make the plan attractive from the standpoint of cost, and benefits derived, is a question for the engineer to decide.

(*c*) A rate of 1 250 000 gal. per acre per day could easily be secured were the Broadway sewage diluted with an equal volume of domestic sewage.

(*d*) The Imhoff tank will remove between 50 and 60% of the suspended solids. This is a good removal for a sewage of this type.

(*e*) The Worcester nozzle was found to give the better distribution of the two types used.

These tests were conducted on sewage from the Broadway Sewer made up of about 750 000 gal. per day of starch works sewage and about 1 500 000 gal. per day of domestic sewage, and having the following approximate analysis as taken into the testing station:

Item.	Parts per million.
Suspended matter	413
Total organic nitrogen	87
Oxygen consumed	200
Bio-chemical oxygen demand	700

In reviewing this report, it was found to be unsafe to estimate the rate of application of 1917 Broadway sewage, including the starch works sewage, at more than 800 000 gal. per day per acre. During the winter of 1916-17, a citizens' committee studied methods of financing the sewage disposal works and finally it drafted and secured the enactment of the Sanitary District Act which became effective in July, 1917.* The first and only bond issue, amounting to $860 000, was favorably voted, February 24, 1920. All other expenditures have been made out of annual taxes.

In November, 1917, a committee of the City Council reviewed the status of sewage disposal, visited about twelve sewage treatment plants in the East, and reached the following conclusions:

(*a*) From the processes observed at the sewage filter plants visited, the Commissioners felt that sand filters and contact beds were unsuitable to conditions at Decatur because of the large area required.

(*b*) That any process involving the use of lime as a precipitant would be unsuited to conditions because of the large quantity of lime required and the large volume of sludge which must be disposed of by some means.

(*c*) That Imhoff tanks and sprinkling filters would be suitable for the domestic sewage and if built of sufficient size would be suitable for the combined sewage and starch factory wastes.

(*d*) That activated sludge might be suitable for the disposal of Decatur sewage provided power could be obtained for less than 1 cent per kw-hr. and that some satisfactory method of dewatering, or disposing of, the sludge could be found.

The report then recommended the employment of consulting engineers and the preparation of plans and specifications.

The Consulting Engineers' Report of 1918.—This report was prepared shortly after the formation of the Sanitary District in 1917, to bring to date in definite form all the various prior investigations. The conclusions of this report were briefly stated, as follows:

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* *Transactions*, Am. Soc. C. E., Vol. 91 (December, 1927), p. 441.

> (*a*) The seriousness of river pollution at Decatur calls for remedial measures. Decided odors are produced, particularly adjacent to the city, but which continue in lessening degree 20 miles or more below the city.
>
> (*b*) The project requires the construction of a high-level intercepting sewer to discharge at its lower end about 19 ft. above mean low water.
>
> (*c*) The results of operation of the testing station (1917) indicate that the mixed domestic and industrial sewage can be treated on sprinkling filters. This type of treatment plant, therefore, can be considered reliable. The rate of treatment remains to be more closely fixed on a larger scale.
>
> (*d*) Water storage is recommended as necessary for an additional water supply. Together with river regulation, it is a helpful factor which will materially improve conditions in the river during extremely dry weather.

The report also included an appendix on subsidence resulting from coal mining. As a result, the coal rights were purchased along nearly 3 500 ft. of right of way for $7 500.

Coal Mine Subsidence Report.—In July, 1918, Professor L. E. Young, of the University of Illinois, submitted a report on ground subsidence under structures proposed by the Sanitary District, with recommendations, as follows:

> (*a*) If possible, arrangements should be made to defer the construction of the Riverside Branch for a period of several years so that the coal under this section may be mined. Sewer construction should not begin at this point until three years after the coal directly beneath has been mined.
>
> (*b*) On the basis of the reported market value of coal rights in this District it is recommended that a coal reservation be acquired along the line of the 1918 report. This should be a strip not less than 500 ft. wide.
>
> (*c*) If the construction is to begin within three years, proper steps should be taken immediately to stop coal mining beneath those portions of the line which will be built.

Notes on Starch Works Sewage, 1919.—In the latter part of 1918, the owners of the starch works proposed to enlarge their plant for the manufacture of glucose, dextrine, and other corn products; and the City started preliminary studies for a large water-impounding project. Therefore, a review of the situation with special reference to the starch works sewage was made, and analyses of this sewage from various sources were summarized, as shown in Table 5. The report concluded that, based on a maximum grind of 25 000 bushels per 24 hours, not more than double the area of sprinkling filters would be required because of the starch works sewage, as compared with domestic sewage.

During the summer of 1919, some tests of the activated sludge process on starch works sewage at the plant were undertaken by the owners. The test was run for 77 days on gluten settler waste in a wooden tank 10 ft. in diameter and 10 ft. deep, holding 5 000 gal. A test of 4 days with straight waste gave no results. The sewage frothed over the tank and no sludge or nitrifying organisms were gained. The waste was then mixed with 4% of city sewage and later with an equal volume of city water with some better results. A test was finally run with a mixture of the waste with two volumes of city water. This mixture had a temperature between 78 and 86° Fahr. The operation was established on the following basis:

	Time, in hours.
Filling	1.00
Aeration	10.00
Settling	3.0
Draining (dewatering)	1.0

The quantity of air was not measured. The ratio of filtros plate area to tank surface was 1 to 4.75, and the air was estimated at 2.3 cu. ft. per min. per sq. ft. of tank surface. Frothing ceased after 6 to 8 hours. The analytical results of a week's operation are given in Table 6.

TABLE 5.—ANALYSES OF STARCH WORKS SEWAGE.*

Items.	RECORDS OF VARIOUS ANALYSES, PARTS PER MILLION.				
	(1).	(2).	(3).	(4).	(5).
Solids:					
Total	290.0	4 860.0	5 866.0	....	5 000.0
In solution	......	3 630.0	2 133.0	....	3 500.0
In suspension	......	1 230.0	2 733.0	1 981.0	1 500.0
Nitrogen:					
Total organic	121.0	478.0	590.0	534.0	138.7
Free ammonia	3.4	1.7	1.0	1.3	1.3
Nitrites	0.0	0.0	0.02	0.01	0.0
Nitrates	2.65	1.50	0.0	0.75	1.5
Oxygen consumed	552.0	......	1 925.0	1 592.0	1 000.0
Biological oxygen	780.0	......	....	....	1 600.0
Alkalinity[6]	......	100.0	....	100.0	100.0
Acidity[7]	Slightly	520.0	....	520.0	520.0

* The crude starch works sewage contains 4 581 parts per million of total solids, of which 4 070 are in solution and 511 in suspension.

NOTES.—(1) Pearse and Greeley, April, 1915, Report, sample, June 20, 1914; (2) Lee's sample, November 24, 1916; (3) Lee's sample, September 1, 1916; (4) Lee's Testing Station Report data for 1916–17; (5) typical analysis used in computations for the report; (6) alkalinity to methyl orange; and (7) acidity to phenolphthalein.

The percentage of total solids recovered as sludge was 35. After being dried and ground, this had the following analysis:

Item.	Percentage.
Moisture	4.3
Total nitrate (N)	7.18
Potash (K_2O)	0.62
Phosphates (P_2O_5)	2.91

Dorr-Peck Experiments, 1920.—In July, 1920, Mr. F. H. Rhodes submitted a report, based on several weeks of experiments with bottles. He stated that the Dorr-Peck process of sewage treatment was applicable to the mixed sewage at Decatur, that 4 parts per million of Fe_2O_3 was helpful, and that 1 000 000 gal. of sewage produced 1.2 tons of sludge, containing 6.35% of ammonia.

TABLE 6.—TESTS OF THE ACTIVATED SLUDGE PROCESS.

Item.	Results, in Parts per Million.	
	Influent.	Effluent.
Total solids	1 592	1 037
Solids in solution	1 377	918
Solids in suspension	215	119
Nitrogen as:		
Total organic	125	23.5
Ammonia	0.2	55.6
Nitrites	......	9.1
Nitrates	......	0.4
Oxygen consumed	225	37.9
Stability (methyl blue)	......	71%

Survey of Industrial Sewage.—From time to time since 1924, the engineers and the superintendent have investigated and reported on some of the minor industrial sewages. A small packing house, in a low-level area and with an average kill of 500 hogs and 70 cattle per week, was studied especially with reference to the operation of a small pumping station. The packing house was required to install a settling tank and grease basin on its sewer outlet. The chemical data obtained during these investigations are summarized in Table 7. All samples were 8-hour composites.

TABLE 7.—RESULTS, IN PARTS PER MILLION, OF ANALYSES OF INDUSTRIAL SEWAGES.

Date of collection.	Source.	Suspended matter.	Total solids.	Total nitrogen.	Ammonia nitrogen.	Nitrates and nitrites.	Stability.*	Oxygen consumed.	Dissolved oxygen.	Bio-chemical oxygen demand, 5 days, 20° cent.	SO_4
10/14/25	Wabash Shops	55	440	0.2	....	0.0	68	17	5.1	17	411
10/15/25	Premier Malt Co., strong waste	27 000	32 400	310	....	0.0	0.0	9 840	0.0	20 000	....
10/15/25	Premier Malt Co., combined waste	426	1 040	51	....	0.1	0.0	278	0.0	600	....
10/16/25	Faries Mfg. Co.	442	3 540	9	6.	23.0	90	103	1.2	13	802
10/16/25	Mueller Mfg. Co., plate room	265	549	0.8	....	0.0	90	23	7.8	50	125
10/16/25	Mueller Mfg. Co., oil wash	557	723	2.0	....	35.0	90	82	6.2	308	314
10/16/25	Danseizen's Packing Co.-no killing	508	......	2.6	16.	....	....	207	....	......	....
10/16/25	Danseizen's Packing Co.-killing	724	3 550	440	....	1.5	0.0	674	0.0	1 393	....
10/20/25	Young's Packing Co.	1 660	4 190	204	....	1.0	0.0	708	0.0	693	....

* Percentage of stability to methylene blue.

NOTE.—Population equivalent calculated from bio-chemical oxygen demand and volume of waste:

Wabash Shops	Negligible
Premier Malt Co	3 000
Faries Mfg. Co	Negligible
Mueller Mfg. Co	Negligible
Danseizen's Packing Co	7 000
Young's Packing Co. (estimated)	3 000
Total	13 000

With the completion of the water-impounding project and the intercepting sewer, it became advisable to make a comprehensive general sewer plan for the entire city providing for new and relief sewers. The city was expanding and the newly built-up areas required sewerage. Many of the existing sewers were overloaded and needed relief. The City and District officials desired to have such sewers properly related to the sewage disposal and river improvement works. A general plan was prepared showing about six major sewer projects, with a total estimated cost of $2 175 200. One of the projects, amounting to about $360 000, was completed in 1927; and plans and specifications were ready for taking bids on another estimated to cost more than $600 000.

As a result of all this effort, intercepting sewers and sewage treatment works have been completed and put into operation with an equivalent population capacity, under conditions at Decatur, of about 150 000. The owners of the starch works are completing process adjustments reducing the quantity and strength of its sewage to an approximate population equivalent of 1 000 per 1 000 bushels of grind. Construction work was first started in 1919, when the upper section of the intercepting sewer was let; and was completed in October, 1927, when the pre-aeration plant was put into operation.

Design Data

Sources.—The progressive design and construction of the Decatur sewage disposal projects over so many years, has made it possible to secure some design data from operating records, as, for instance, sewage quantities and characteristics at the treatment plant. In the first place (1914), weirs were built at the main sewer outlets and the rates of flow measured. As already noted, much design data came from the operation of the testing stations. The remainder was secured through the usual routine of local investigation.

Population.—The first population forecast and distribution of population over the city was made in 1915, and was reviewed in 1919 and 1925. The basis of design of works easily enlarged was 60 000, and of intercepting sewers and structures not easily enlarged, 120 000. An additional allowance was made for the sewage of major industries.

Quantities of Sewage.—The capacity of the intercepting sewer varies for different sections, as shown on Fig. 2. In the upper section where separate sewers are to be built, the capacity flowing full is for sewage at the rate of 1 000 gal. per capita per 24 hours from 20 000 people. This allowance includes capacity for a considerable industrial area. Then follows a short section taking combined sewage from about 250 acres located above the dam with a capacity of 5 750 gal. per capita per 24 hours from 28 700 population. There is an overflow just below the dam, and the capacity of the next section, which is largely in tunnel, is for sewage at the rate of 730 gal. per capita per 24 hours from 28 700 people. The next section, which is through a built-up part of the city, provides capacity for 950 gal. per capita per 24 hours from 68 000 people. The final section built largely in open country and, therefore, quite easily duplicated, has a capacity of 40 700 000 gal. per day equivalent to 340 gal. per capita from 120 000 people.

The first installation of sewage treatment works (Imhoff tanks, sprinkling filters, and appurtenances) was given a capacity for 60 000 people and 7 960 000 gal. per day, equivalent to 133 gal. per capita per 24 hours. This included 1 250 000 gal. per day of starch works sewage, equivalent to about 21 gal. per capita per 24 hours. When the additions to the sewage treatment plant were designed in 1926, the rates of flow (see Table 8) were estimated largely on the basis of meter records for about two years at the treatment plant.

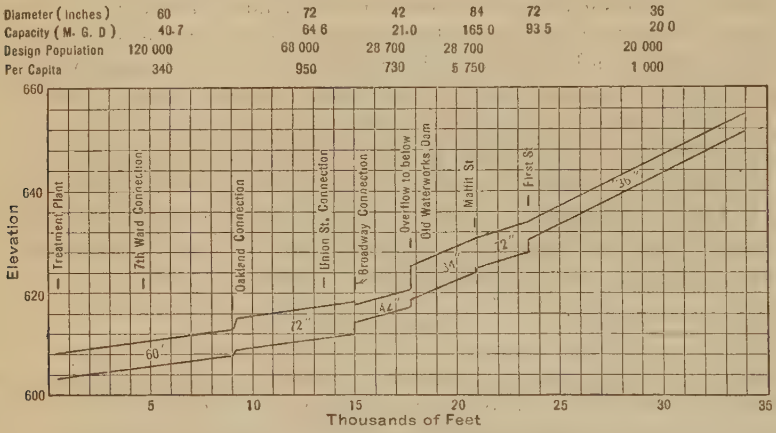

Fig. 2.—Profile of Intercepting Sewers, Decatur, Ill.

Including industrial sewage, the average dry season rate of flow is 167 gal. per capita per 24 hours from 60 000 people. It has been difficult to estimate the quantity of sewage from the starch works because of the rapid increase in the grind and the process adjustments resulting in changing quantities of sewage per bushel of grind.

TABLE 8.—Rates of Flow Estimated in 1926.

Item.	Million Gallons per 24 Hours.	
	Wet season.	Dry season.
Average	14.0	10.0
Maximum	16.0	....
Minimum	....	5.2

Sewage Characteristics.—A determination of the strength and character of Decatur sewage has been a most difficult element in the design. The domestic sewage is mixed with a variable industrial sewage from the starch works. This industrial sewage has increased with the grind from a population equivalent of 75 000 more or less during the period of 1914 to 1917, to one of more than 300 000 in 1926. In addition, the grind and the sewage vary with business conditions, the grind having recently ranged in a single year from 20 000 to 45 000 bushels per day. The owners of the starch works, under the direction of Dr. Edward Bartow, have undertaken process adjustments to try and recover products which, until recently, were lost to the sewer, with a marked reduction in the strength of their sewage. The sewage first increased in strength with the growth at the starch works; and then decreased with the reduced waste.

CONSTRUCTION PROJECTS

General.—Construction work has proceeded almost continuously from 1919 to 1927 with a total expenditure chargeable to construction, including engineering, of $1 666 783, and involving about fifteen general projects. Engineering supervision and inspection on these projects and during this time has equalled about 6.0% of the total amount of the final estimates.

The land for the sewage treatment plant is about 57 acres in area and cost $7 419. Rights of way for the intercepting sewer cost $11 702, of which $7 500 was for the purchase of coal rights under the intercepting sewer. The total payment for right of way amounts to about $1 per lin. ft. of sewer.

Embankment for Intercepting Sewer.—The first construction contract was for the embankment at the lower end of the intercepting sewer (1919). The embankment is about 2 900 ft. long and 12 ft. deep below the invert of the sewer at the point of maximum depth. It has a top width of 19.5 ft. for part of its length and 26.5 ft. for the remainder, with side slopes of 1 on 1.5. It was built of a somewhat sandy yellow clay put down in 12-in. layers and compacted by the travel of the teams. It was allowed to stand about 10 months before further use. After 5 years of service, there have been no signs of settlement. The sides have been planted with hay, clover, etc., and there has been no need of repairing them. The original contract comprised about 35 000 cu. yd. of embankment at $0.34 per cu. yd. in place. All the material was taken from a borrow-pit near the right of way.

Intercepting Sewers.—The intercepting sewer built by the Sanitary District is 18 317 ft. long and cost $598 000, or an average of $32.60 per ft. In the three contracts under which it was built, alternate bids were taken on several types of construction and the least expensive type was selected with results, as follows:

Type.	Approximate length, in feet.
Monolithic concrete	5 700
Segmental block	8 700
Reinforced concrete pipe	3 600

Sewage Treatment Works.—The sewage treatment works were started in 1921 with the construction of the treatment plant by-pass. This was a 48-in. concrete sewer, 1 131 ft. long, including an overflow chamber, an outfall structure, and appurtenances. The sewer cost $20.20 per ft.

Other construction projects at the sewage treatment plant were a roadway, 3 100 ft. long; an elevated tank with a capacity of 50 000 gal.; a ground-water supply; a sludge locomotive; and an electric power line. The total amount spent in connection with these various projects was $653 143, including engineering. Except for a rather liberal sludge capacity in the Imhoff tanks (3.03 cu. ft. per capita), the works so built have a capacity for a strong domestic sewage from about 60 000 people, indicating a cost of $10.92 per capita.

In the spring of 1925, a small pre-aeration testing station was put into operation; and during 1926 and the first half of 1927, the sewage treatment plant was enlarged by the addition of pre-aeration tanks and appurtenances.

South Main Street Pumping Station.—A small low-level area of about 175 acres is served by the South Main Street Pumping Station. The area includes two small packing houses with an average kill of about 500 hogs and 70 cattle per week. The packing-house sewage passes through a small grease basin before reaching the pumping station. The pumping station contains bar screens with $\frac{5}{8}$-in. clear openings and two 4-in. vertical centrifugal pumps with motors operated by floats. The station is automatic in operation and is visited once a day for inspection, cleaning screens, and oiling. This station cost $12 454 to build, and has an installed pump capacity of 1 000 000 gal. per day.

Pre-Aeration Testing Station.—During February and March, 1925, a small testing station (Fig. 3) was built to study the effect of partial or pre-aeration on sprinkling filter rates. The major items of construction are briefly described as follows:

(*a*) An aeration tank 12 ft. in diameter, 10.0 ft. liquid depth, and containing 6 570 gal. At 2 hours' displacement, this tank had a capacity for 78 000 gal. of sewage per 24 hours. It was fitted with a surface aerator or agitator of the Simplex type. The agitator wheel was 36 in. in diameter and operated by a 1-h.p. motor at 80 rev. per min.

(*b*) Originally, there were two settling tanks, each 5 ft. in diameter and 9 ft. liquid depth. At a sewage flow of 78 800 gal. per 24 hours the displacement period was 0.63 hours, with an area for about 2 000 gal. per sq. ft. per 24 hours, with both tanks in operation. These rates were found to be too high and a single tank, 10 ft. in diameter and 10 ft. liquid depth, was added. This provided a displacement period of 1.5 hours, with an area for about 670 gal. per sq. ft. per 24 hours, with all three tanks in service.

(*c*) The sludge re-aeration tank had a capacity of 810 gal. With 10% return sludge (8 000 gal. per 24 hours), the aeration period was about 2.5 hours.

(*d*) The sprinkling filter was 14 ft. in diameter and 6 ft. 4 in. deep, with one Taylor nozzle. It was filled with 1 to 2-in. stone from the large sprinkling filter; and dosed from a small tank controlled at first by a motor-driven butterfly valve, and, later, by an automatic siphon.

(*e*) In addition to the aeration tank, settling tanks, re-aeration tank, and sprinkling filter, the testing station included a blower, an air meter, a small pump to serve the sprinkling filter, a number of orifice boxes, and other appurtenances.

It took about a month to build the station and the costs were approximately as follows:

Materials	$3 230
Labor	800
Engineering	652
Total	$4 682

Covering Grit Chambers, Conduits, Gas Vents, Etc.—The starch works sewage at Decatur contains very substantial quantities of sulfur dioxide and soluble organic matter which, with the domestic sewage, decomposes to form hydrogen sulfide. This decomposition is greatly accelerated by the relatively high temperature. To reduce such odors, concrete covers were built during the summer of 1926 over the grit chamber and the conduits and gas vents of

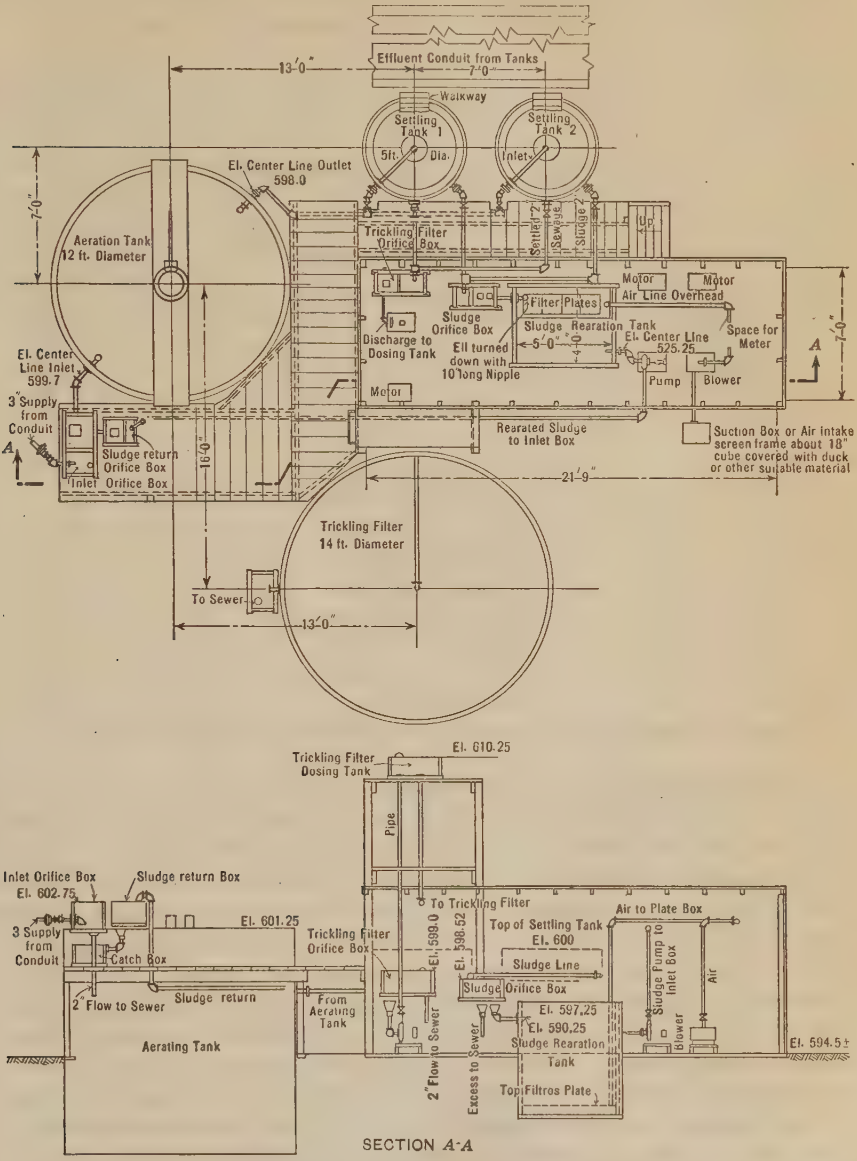
Effluent Conduit from Tanks
13′0″
7′0″
Walkway
Settling Tank 1
5ft. Dia.
Settling Tank 2
Inlet
El. Center Line Outlet 598.0
7′0″
Aeration Tank 12 ft. Diameter
Trickling Filter Orifice Box
Settled 2
Sewage
Sludge 2
Up
Motor
Motor
Air Line Overhead
Filter Plates
Sludge Orifice Box
Sludge Rearation Tank
Discharge to Dosing Tank
Ell turned down with 10″long Nipple
5′0″
4′0″
El. Center Line 525.25
Space for Meter
Pump
Blower
A
7′0″
El. Center Line Inlet 599.7
3″Supply from Conduit
A
Motor
Sludge return Orifice Box
Inlet Orifice Box
16′0″
Rearated Sludge to Inlet Box
Suction Box or Air intake screen frame about 18″ cube covered with duck or other suitable material
21′9″
Trickling Filter 14 ft. Diameter
To Sewer
13′0″
Trickling Filter Dosing Tank
El. 610.25
Pipe
Inlet Orifice Box El. 602.75
Sludge return Box
3″Supply from Conduit
El. 601.25
Catch Box
2″Flow to Sewer
Sludge return
Aerating Tank
Trickling Filter Orifice Box
From Aerating Tank
To Trickling Filter
El. 599.0
El. 598.52
Top of Settling Tank El. 600
Air to Plate Box
Sludge Line
Sludge Orifice Box
El. 597.25
El. 590.25
Sludge Rearation Tank
Top Filtros Plate
Sludge Pump to Inlet Box
Air
Blower
2″Flow to Sewer
Excess to Sewer
El. 594.5±
SECTION A-A

FIG. 3.—GENERAL PLAN, TESTING STATION, DECATUR, ILL.

the Imhoff tanks. The spaces under the covers of the grit chamber and conduits were connected by pipes to a suction fan which delivered the confined air and gases to a small brick furnace in which they were burned with the gases from the digesting sludge. The covers over the gas vents are shown in Fig. 4. The cost of this work, including engineering, was about $15 000.

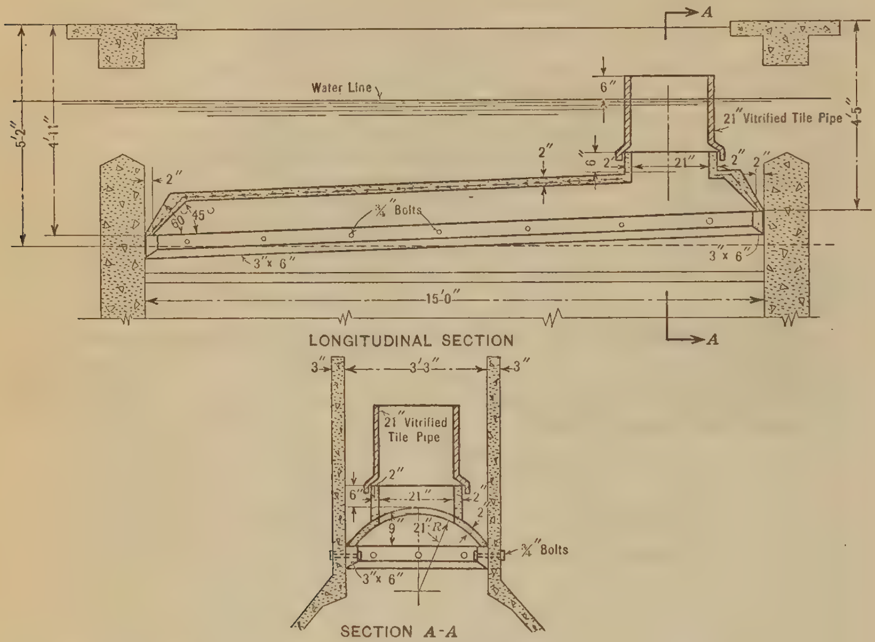

FIG. 4.—DETAILS OF GAS COLLECTOR, SEWAGE DISPOSAL WORKS, DECATUR, ILL.

The sludge beds as built covered an area of 40 000 sq. ft., which was found to be too small as the grind at the starch works increased. Therefore, in 1926, a contract was let for building 16 500 sq. ft. of additional beds at a cost of $9 000, which is equivalent to $0.55 per sq. ft. of sludge bed. The filtering material in the sludge beds comprises 1½ in. of fine sand, 3 in. of coarse sand, 4 in. of roofing gravel (¼ to ⅜ in. in size), and below this a layer of graded broken stone or gravel up to 2-in. size about the under-drains.

During the fall of 1926 and until the autumn of 1927, a pre-aeration plant was built. This plant comprises six aeration tanks and two settling tanks of the Dorr type, all with a rated capacity of 10 000 000 gal. per day.

As nearly as may be determined from the records of the Sanitary District, the total expenditures for construction projects have amounted to $1 666 783 (see Table 9). Active construction work started just after the World War in 1919 and was completed in 1927, covering nine construction seasons. This is a total cost of $33.30 per capita, based on 50 000 population for the period of construction.

TABLE 9.—SUMMARY OF EXPENDITURES FOR CONSTRUCTION PROJECTS.

Item.	Amount.
Embankment for intercepting sewer, 1919	$53 620
Intercepting sewer, Contract 2	135 131
Intercepting sewer. Contract 3	329 205
Intercepting sewer, Contract 4	134 365
By-pass sewer	22 894
Sewage treatment plant, Contract 1	570 707
Pre-aeration plant, Contract 5	142 785
Blower house, Contract 6	44 663
Equipment for pre-aeration plant	27 832
Wrecking dosing tank	1 100
Seventh Ward sewer connection	19 681
South Main Street pumping station	12 454
Extension of sludge beds	9 000
Sludge locomotive	1 805
Gas-collecting and burning apparatus	15 000
Electric power transmission line	5 255
Roadway to treatment plant	7 724
Land and rights of way	19 122
Equipment of laboratory	2 820
Treatment plant water supply	10 639
Landscaping at treatment plant	971
Engineering (approximate), 1927	100 000
Total	$1 666 783

WATER-IMPOUNDING PROJECT

Relation to Sewage Disposal.—The need for an additional water supply for Decatur became evident as early as 1914. The unregulated flow in the Sangamon River is frequently as low as 5 000 000 gal. per day for several months at a time. On one occasion only the fortunate and unexplained blowing up of a channel dam up stream let down a sufficient volume of water to tide over the situation. By 1920, the average daily water consumption of the city was 5 540 000 gal. per day, exclusive of the needs of the Staley Manufacturing Company for cooling water. Consequently, a large water-impounding project was undertaken and an earth and concrete dam put under construction in June, 1920.

At that time the Sanitary District was building intercepting sewers, and the Trustees were asked to contribute Sanitary District funds in consideration of the possibility of the diluting and flushing water as an aid to sewage disposal during periods of low flow. Computations, however, demonstrated that there would not be a sufficient quantity of water available from the storage reservoir to supply enough oxygen for the sewage, and no appropriations were made for this purpose.

Taking 500 parts per million as the approximate average oxygen demand of the sewage, a sewage flow at the rate of 10 000 000 gal. per day would require about 42 000 lb. of oxygen per 24 hours. Assuming 5.0 parts per million of dissolved oxygen available from the reservoir water and no measurable flow in the river, it would require a discharge from the reservoir at the rate of 1 000 000 000 gal. per day. Obviously, nothing like this rate of discharge was possible except for very short periods, as the safe yield of the impounding project was estimated at about 35 000 000 gal. per day. Nevertheless, the river near the city has been freshened by the occasional discharge of reservoir water as indicated in Table 10.

Brief Description.—Lake Decatur is formed by an earth and concrete dam across the valley of the Sangamon River. The total length of the dam is 1 715 ft. At the north end is an earth embankment, 510 ft. long. The center section, a solid concrete spillway, 525 ft. long, is 28.5 ft. high and has a width at the base of 72 ft., including the down-stream concrete apron. There is an up-stream clay apron, 28 ft. wide, making the total width of the base of the dam 100 ft. A movable crest is built on top of the spillway by which the water level can be raised 2.5 ft. up to Elevation 612.5.

TABLE 10.—RIVER FLUSHING WITH RESERVOIR WATER.

Date.	Duration of flushing, in hours.	Approximate Rate of Flushing.	
		In million gallons per day.	In cubic feet per second.
1925:			
June 6	22	44	67
July 18	24	44	67
July 28	24	44	67
August 6	22	51	79
August 18	22	51	79
August 25	21	51	79
September 9	24	51	79
1926:			
July 2	26	73	112
July 16 to 30	Intermittent	177	272
August 16 to 20	"	177	272

Extending under 1 500 ft. of the length of the dam is a line of sheet-piling largely of steel, but with wood sheeting at either end. Under the concrete part of the dam (see Fig. 5), the steel sheet-piling extends down about 30 ft. to a firm connection with a layer of hardpan glacial till. At the north end of the spillway, a gate chamber has been provided so that 1 000-h.p. water turbines can be installed if desired.

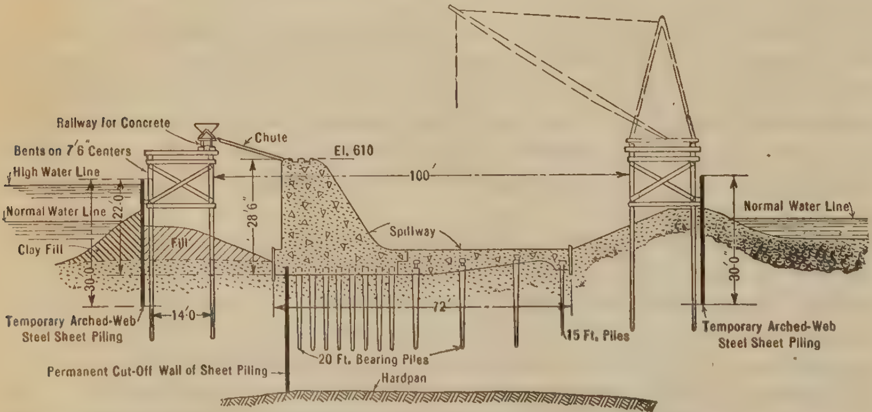
Railway for Concrete
Chute
Bents on 7′6″ Centers
El. 610
High Water Line
Normal Water Line
22′0″
28′6″
100′
Clay Fill
Fill
Spillway
Normal Water Line
30′0″
14′0″
72′
30′0″
Temporary Arched-Web Steel Sheet Piling
15 Ft. Piles
Temporary Arched-Web Steel Sheet Piling
20 Ft. Bearing Piles
Permanent Cut-Off Wall of Sheet Piling
Hardpan

FIG. 5.—CROSS-SECTION OF COFFER-DAM FOR CONCRETE SPILLWAY OF DAM, DECATUR, ILL.

The dam was built on a sandy gravelly formation underlying the valley soil and overlying the Wisconsin and Illinoisan tills. The Wisconsin till (or drift) is of terminal moraine origin and the Illinoisan of ground moraine structure. Thus, the Illinoisan till is very hard and dense with a maximum content of impervious clay. For the most part the steel sheeting was driven into this formation effecting a very good cut-off.

The reservoir back of the dam covers about 3 000 acres and extends up stream about 12 miles, with an average width of somewhat less than ½ mile. The volume of the reservoir is from 6 000 000 000 to 8 000 000 000 gal. and the average depth 7.5 ft. About one-fourth of the area was timbered and had to be cleared. Nearly fifty buildings were removed and 17 miles of marginal and cross-highways were raised, relocated, and protected with paving.

Method of Financing.—The water-impounding project cost more than $2 000 000. The City had available the proceeds of two bond issues amounting to somewhat more than $600 000. Money from annual taxes was appropriated for construction purposes, amounting to more than $250 000. It was necessary, therefore, to raise about $1 300 000 in some other way. The plan adopted was to form a local water company of Decatur citizens that was to issue stock and notes secured by the reservoir land and by a contract with the City to pay the water company enough to provide a 7% rate of interest and to retire the stock in 16 years. This stock in the amount of $1 000 000 was over-subscribed in a short campaign, and the necessary funds were secured. By other subscriptions and loans this amount was raised to more than $1 300 000.

The cost of the water-impounding project may be summarized, approximately, as follows:

Item.	Amount.
Land for dam	$30 000
Dam	815 000
Reservoir land	610 000
Clearing	115 000
Roads and bridges	400 000
Road embankment protection	200 000
Engineering for dam	40 000
Surveys and miscellaneous	50 000
Total	$2 260 000

The dam was built under a cost-plus contract, with a participation clause and an upper limit of $1 025 000. The unit costs of the major items were stated in the contract, as follows:

Item.	Amount.
Stripping	$1.10 per cu. yd.
Earth excavation above Elevation 595.0	0.52 " " "
Earth excavation below Elevation 595.0	0.92 " " "
Earth borrow	0.52 " " "
Rock excavation	3.00 " " "
Earth embankment, Class A	0.17 " " "
Earth embankment, Class B	2.30 " " "

Item.	Amount.
Placing loam	$1.70 per cu. yd.
Seeding	0.06 " sq. yd.
Drainage for slope paving	0.23 " lin. ft.
Slope paving	1.55 " sq. yd.
Steel sheet-piling, Class A	0.07 " lb.
Steel sheet-piling, Class B	0.06 " "
Wooden sheet-piling	1.05 " sq. ft.
Concrete in spillway	12.20 " cu. yd.
Concrete in gate setting	17.00 " " "
Concrete in apron	9.20 " " "
Concrete in toe-wall	15.00 " " "
Concrete in abutments	13.70 " " "
Concrete in conduit	25.00 " " "
Reinforcing steel	0.06 " lb.

Notes on Starch Works

Growth of Starch Works.—In 1912, the cornstarch factory began operating. The capacity of the plant at that time was about 10 000 bushels of corn per 24 hours which, by 1915, had increased to 15 000 bushels. Then, due to the World War, the plant was practically closed until 1917, when regular operation was resumed. Since that time it has operated almost continuously with a progressively increasing grind to as much as 45 000 bushels per 24 hours. The products of manufacture have lately been diversified, so that at present various types of cornstarch, gluten meal, dextrine, glucose, corn sugar, corn oil, soy bean oil, and soy bean meal are produced. The Company employs about 2 000 hands when operating near the rated capacity.

The rather phenomenal growth was entirely unanticipated, even by the officials of the industry itself. In 1918 when plans for the Decatur sewage treatment plant were being formulated, the grind was less than 15 000 bushels of corn per 24 hours. The Company fully stated its plans for the future, which anticipated a maximum daily grind of 25 000 bushels and the erection of a plant for converting wet starch into glucose and corn sugar. The glucose plant was built to take advantage of the market prices of dry starch and glucose so as to secure the best gross revenue. This plant did not in itself increase the corn-grinding capacity of the starch plant, nor was it anticipated that it would. In reality, however, it has been the cause of doubling the anticipated corn grind, through better adjustment to the market and a consequent growth of business.

Plant Processes and Wastes.—Briefly, the process of manufacture is as follows: (*a*) Steeping the corn in a dilute sulfur dioxide solution for a number of hours; (*b*) crushing the steeped corn and separating the germ from the hull; (*c*) grinding the hull, and separating the husks from the ground meal; (*d*) separating the starch and gluten by sedimentation and flotation; (*e*) washing and drying the starch and gluten; and (*f*) converting wet starch to glucose and sugar.

A certain amount of liquid waste results from each operation. In some cases the wastes are re-used in the plant for carrying purposes and thus return into the process; in others, the liquids are evaporated, and the concentrated materials are mixed and dried for the market. The sequence of the various processes is indicated in Fig. 6.

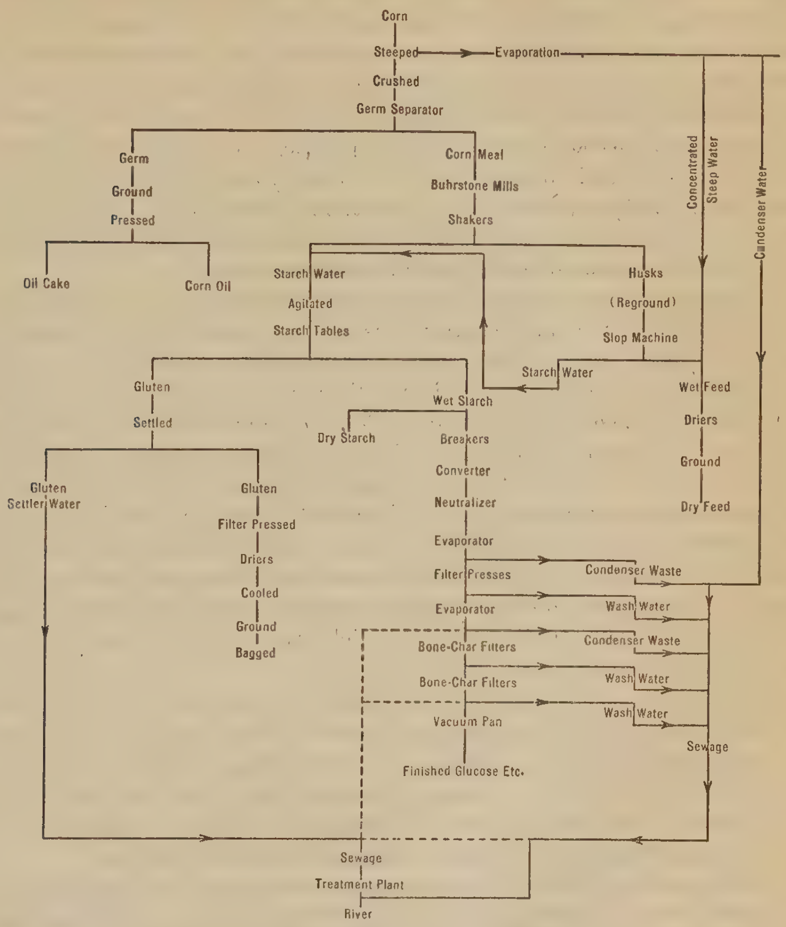
Corn
Steeped
Evaporation
Crushed
Germ Separator
Germ
Ground
Pressed
Oil Cake
Corn Oil
Corn Meal
Buhrstone Mills
Shakers
Concentrated Steep Water
Condenser Water
Starch Water
Agitated
Starch Tables
Husks
(Reground)
Slop Machine
Starch Water
Gluten
Settled
Wet Starch
Dry Starch
Breakers
Converter
Neutralizer
Evaporator
Filter Presses
Evaporator
Bone-Char Filters
Bone-Char Filters
Vacuum Pan
Finished Glucose Etc.
Wet Feed
Driers
Ground
Dry Feed
Gluten Settler Water
Gluten
Filter Pressed
Driers
Cooled
Ground
Bagged
Condenser Waste
Wash Water
Condenser Waste
Wash Water
Wash Water
Sewage
Sewage
Treatment Plant
River

FIG. 6.—GRAPHICAL REPRESENTATION OF PROCESSES AND SOURCES OF STARCH WORKS SEWAGE, DECATUR, ILL.

There are, of course, in a plant of such size, many wastes, but those of major importance are: (1) Gluten settler water, resulting from Process (*d*), which contains almost all the soluble organic and mineral waste matter from the corn; (2) condenser water from the steep water evaporation pans, which contains considerable organic entrainment; (3) condenser water from the glucose plant, which is relatively free from entrainment; and (4) wash water from cleaning out condenser pans, bone filters, filter cloths, etc.

The maximum quantities of sewage expected from the starch plant was 50 gal. of strong sewage and 200 gal. of condenser water per bushel of corn ground. These were the quantities on which the design of the original sewage treatment plant was predicated, with the expectation that the condenser water would be by-passed direct to the river if desired.

The sewage treatment plant was placed in operation in May, 1924, and during its initial running the total quantity of mixed sewage was between 15 000 000 and 17 000 000 gal. per 24 hours. This comprised about 5 000 000 gal. per day of domestic sewage from the city and from 10 000 000 to 12 000 000 gal. per day of industrial sewage from the starch works. Investigation showed (*a*) that the starch works were grinding about 35 000 instead of the anticipated 25 000 bushels of corn per day; (*b*) that the Company's sewers were arranged so that the 6 000 000 to 10 000 000 gal. of condenser water could not be separated from the strong sewage for diversion into the river; and (*c*) that the private 24-in. sewer, through which the condenser water was to have been diverted, was of insufficient capacity.

Investigations and analyses of the glucose condenser water showed that it was practically of the same character as the raw water before entering the condenser, except for occasional entrainments of glucose or sugar. By careful operation of the evaporation pans it was thought that this entrainment could be reduced to a negligible minimum. The Company, therefore, was allowed to construct a conduit for returning this condenser water to the impounding reservoir. This was completed and placed in operation in July, 1925, and relieved the sewage treatment plant of from 6 000 000 to 10 000 000 gal. per day of relatively hot condenser water, the volume depending on the season of the year and the temperature of the cooling water from the lake. This diversion reduced the volume of sewage received at the treatment plant to about 11 000 000 gal. per day, 50% of which was from the starch works and the remaining 50% from the city.

Characteristics of Mixed Sewage.—The five-day bio-chemical oxygen demand of the week-day city sewage, when the starch works is not in operation, is about 125 parts per million. The average monthly bio-chemical oxygen demand of the mixed sewage, including that from the starch works, varies directly with the corn grind and inversely with the volume of sewage, and has ranged from 500 to 800 parts per million in dry weather.

The temperature of the mixed sewage as received at the treatment plant has varied from 70 to 104° Fahr. during winter and summer, respectively. These very warm temperatures caused rapid decomposition of the mixed sewage, so that when it reached the sewage plant, hydrogen sulfide odors resulted. These odors were intensified after the condenser water was removed from the sewers, due to the greater concentration of the mixed sewage so that paint on houses a mile from the plant was discolored.

Population Equivalent and Its Reduction.—Analyses of the city sewage and the starch works sewage at this time indicated a total population equivalent of about 350 000 for the mixed sewage as received at the treatment plant, of which about 300 000 was contributed by the starch works. These figures were obtained by dividing the total pounds per 24 hours of 5-day bio-chemical oxygen demand by 0.17 lb. as the per capita contribution of

5-day bio-chemical oxygen demand.* This factor seems to apply satisfactorily to the Decatur domestic sewage population equivalent when the starch works is shut down. The estimated human population connected to the sewers is about 40 000 and the minor industrial population equivalent, exclusive of the starch waste, is between 10 000 and 15 000. (See "Survey of Industrial Sewage," page 554.)

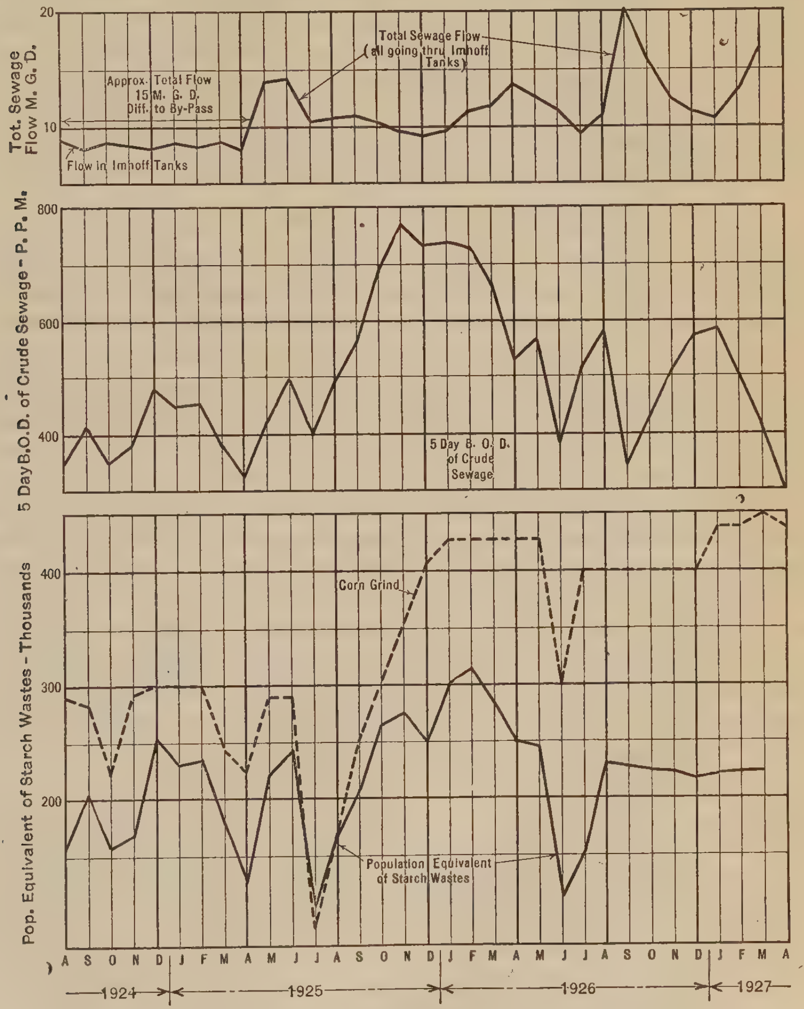

FIG. 7.—STARCH WORKS DATA.

On Fig. 7 is shown the quantities of sewage, in million gallons per day, the 5-day bio-chemical oxygen demand, in parts per million, the corn grind, in bushels per 24 hours, and the population equivalent of the starch works sewage for 1924-27. The population equivalent of the starch works sewage was calculated by subtracting 50 000 from the total population equivalent as determined from the regular daily analyses of composite samples of the mixed sewage received at the sewage treatment plant.

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* Calculated from average of eight cities, U. S. Public Health Service *Bulletin 132*, pp. 35-111.

These curves show that the Company has improved its operation and decreased its losses as indicated by the maximum population equivalent of 370 000 in February, 1926, and the average population equivalent of 275 000 for January, February, and March, 1927, when the grind of corn was higher than in February, 1926. Further reductions are expected in the near future that will bring the population equivalent of the starch works sewage to less than 100 000, thus greatly relieving the burden now imposed on the city sewage treatment plant. At present (November, 1927), the population equivalent of the starch works sewage is about 260 000.

SEWAGE TREATMENT WORKS, FIRST INSTALLATION

General Arrangement.—In 1922, the Board of Trustees authorized the preparation of plans and specifications for the sewage treatment plant comprising settling tanks of the Imhoff type, sprinkling filters, and appurtenances. The rated capacity of the plant was 60 000 people and an average dry-season sewage flow at the rate of 8 000 000 gal. per day. It was expected that a higher rate of flow might occur for several weeks at a time during wet seasons. Included in the quantity of 8 000 000 gal. per day was an allowance of 1 250 000 gal. per day for sewage from the starch works.

The sewage treatment plant is built on 57 acres of low land, about 2.5 miles from the business center. First-class houses are within 2 000 ft. and closer lots are being steadily developed. The general plan of the first construction is shown in Fig. 8. The site is surrounded by a flood dike with its top 19 ft. above mean low water in the river. The high water of 1926 just reached this elevation. The general plan provided for doubling the capacity or for an ultimate capacity of 120 000. The first installation included only 3.0 acres of sprinkling filter, or one-half the area needed for the stated capacity of 60 000 and the 1 250 000 gal. per day of starch works sewage.

The elevations of the principal structures were determined as shown in Table 11.

Hydraulic gradients, velocities, and elevations were computed for an average sewage flow of 8 000 000 gal. per day with a minimum flow of 5 400 000 gal. and a maximum of 44 200 000 gal. per 24 hours. Shapes and sizes were designed to maintain approximately the following velocities:

Item.	Velocity, in feet per second.
Grit chamber	1.0
Conduit to settling tanks	1.0
Settling tank influent conduits	1.0
Settling tank effluent conduits	0.5
Sprinkling filter collecting conduits	1.5
Main effluent conduits	2.0

The total loss of head is thus 20.91 ft., of which 1 ft. was included ahead of the settling tanks as a contingent allowance to permit installation of sewage treatment devices not developed at the time of design.

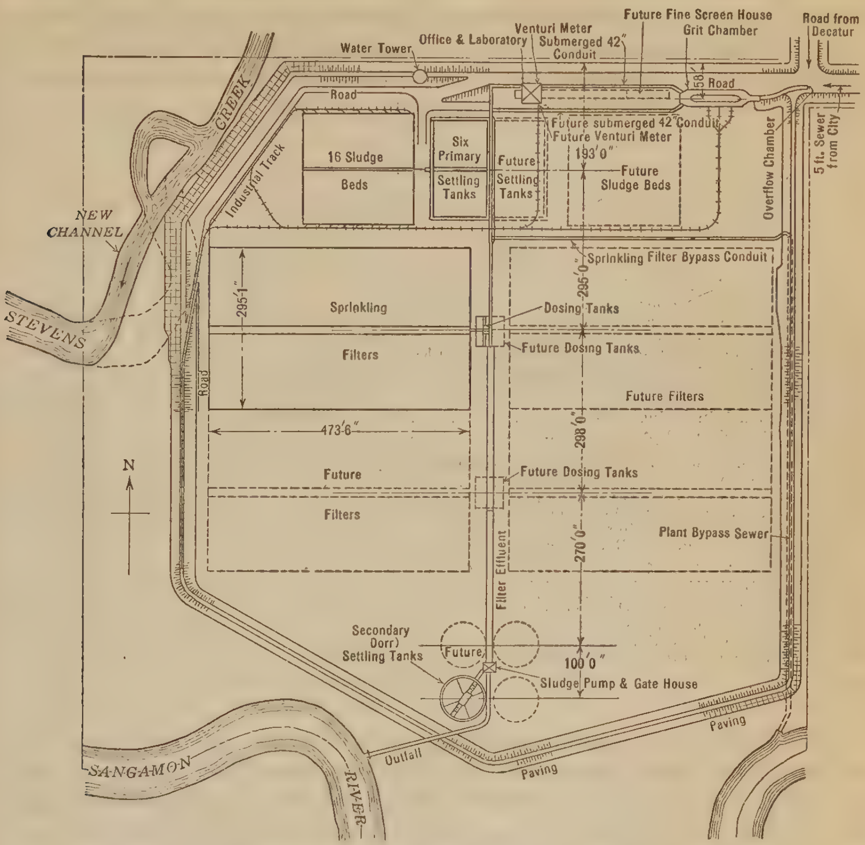
Future Fine Screen House
Road from Decatur
Venturi Meter
Grit Chamber
Water Tower
Office & Laboratory
Submerged 42" Conduit
58'
Road
Road
Future submerged 42" Conduit
Future Venturi Meter
193'0"
CREEK
Industrial Track
16 Sludge Beds
Six Primary Settling Tanks
Future Settling Tanks
Future Sludge Beds
Overflow Chamber
5 ft. Sewer from City
NEW CHANNEL
STEVENS
Sprinkling Filter Bypass Conduit
295'1"
Sprinkling Filters
295'0"
Dosing Tanks
Future Dosing Tanks
Road
Future Filters
473'6"
298'0"
N
Future Filters
Future Dosing Tanks
Plant Bypass Sewer
Filter Effluent
270'0"
Secondary Dorr Settling Tanks
Future
100'0"
Sludge Pump & Gate House
Paving
Outfall
Paving
SANGAMON
RIVER

FIG. 8.—GENERAL PLAN, SEWAGE TREATMENT PLANT, DECATUR, ILL.

Parts of the Plant.—The principal parts of the plant comprising twenty-one items, are listed as follows:

1.—Plant by-pass sewer.
2.—Connecting sewers.
3.—Coarse screen.
4.—Grit chamber.
5.—Venturi meter and house.
6.—Imhoff tanks.
7.—Sludge beds.
8.—Dosing tanks.
9.—Sprinkling filters.
10.—Final settling tanks.
11.—Outlet to river.
12.—Filter by-pass.
13.—Final settling tank by-pass.
14.—Water supply.
15.—Lighting.
16.—Industrial railway.
17.—Drainage system.
18.—Office building and laboratory.
19.—Flood protection.
20.—Planting.
21.—Roadways.

The grit chamber* comprises three flowing-through conduits, each 75 ft. long. Two of these conduits are 8.0 ft., and the other, 10.0 ft., wide, with $1\frac{9}{16}$-in. clear openings. The velocity and depth of flow in the conduits are maintained by a proportional orifice at the outlet end. The floor of the grit chamber stands about $8\frac{3}{4}$ ft. in the clear above the ground level, so that sludge cars on industrial tracks are run in under the chambers for the remaining grit. Each flowing-through compartment has three 18-in. gate-valves controlling openings in the floor through which grit is dumped into the cars.

TABLE 11.

Item.	Elevation, in feet.
Invert of sewer at entrance to plant	602.96
Sewage surface at entrance to plant	604.41
Sewage surface at entrance to settling tanks	603.17
Average elevation of sewage surface in settling tanks	602.61
Sewage surface at outlet of settling tanks	602.13
Sewage surface at first dosing tank	602.12
High sewage mark in dosing tank	601.90
Low sewage mark in dosing tank	595.90
Surface of filter stone at near nozzle	595.10
Surface of filter stone at far nozzle	594.30
Invert of under-drains at northwest corner of filter	588.05
Invert of main collector near first dosing tank	584.78
Sewage surface in main collector near first dosing tank	586.88
Invert of main conduit at final settling tanks	584.32
Sewage surface in final settling tank	586.08
Water surface in Sangamon River, August, 1921	585.4
Low water	583.5
High water	598.5
Top of flood dike	602.5
Center line of settling tanks sludge outlets	598.08
Invert of sludge channel at settling tanks	597.67
Sand surface in sludge beds	592.18
Invert of 12-in. main collector at settling tank end of sludge beds	588.76

A 42-in. cast-iron pressure conduit extends from the grit chamber to the settling tanks. This is designed for a minimum velocity of 1.0 ft. per sec.; and, later, will have to be duplicated. On its way the conduit passes through the basement of the office building where it includes a 42-in. Venturi meter tube, the register and recorder of which are set in the office above.

The settling tanks are of the two-story or Imhoff type. The first installation comprises six units, each 27 ft. 10 in. wide and 96 ft. long, inside the walls at the top. The liquid depth is 27.0 ft. and the free-board is 24 in. In each tank are two flowing-through or settling compartments. There are three gas vents in each of the three tanks having a total width of 9.0 ft. equivalent to 31.1% of the total tank surface.

The six settling tanks provide the following capacities:

Item.	Cubic feet.
Settling	63 700
Sludge	181 900
Neutral zone Scum	83 000

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* *Transactions*, Am. Soc. C. E., Vol. 91 (December, 1927), p. 535.

These volumes provide a displacement period of 1.44 hours in the settling compartments when the flow is 8 000 000 gal. per day, and 3.03 cu. ft. per capita in the sludge compartment.

There are two conduits at each end of the settling tanks and a single conduit along the outside of each of the two end tanks, which provide for reversal of the flow. The sewage flow is controlled by so-called "cone orifice" valves. These have proved to be very satisfactory in operation with relatively uniform and complete use of the displacement capacities of the six tanks. The sewage flows out of each tank over weirs 16 ft. long, with suspended screens in front.

The settling tanks are built of reinforced concrete in two units with a main expansion joint between. The side walls are 20 in. thick at the bottom and are tied together by cross-walls heavily reinforced. The baffles forming the settling compartment were built up by a cement gun shooting on to expanded metal, with resulting smooth surfaces and satisfactory results. Each baffle is divided into three sections by vertical expansion joints over each end cross-wall.

Sludge is withdrawn through 8-in. cast-iron sludge pipes having 4.5 ft. of head on the center line of the outlets. These outlets are controlled by sluice-gates and discharge into open conduits leading to the sludge beds. Each of the tanks is fully equipped with a pressure water system.

The first construction work included 40 000 sq. ft. of sludge bed area with provision for an extension (since made) of 16 500 sq. ft. There were originally sixteen sludge beds equally divided on the two sides of a central sludge conduit. Each bed is 100 ft. long by 25 ft. wide, separated from the others by concrete plank partition 2 in. thick. Industrial track for sludge cars are placed along the center of each bed.

Two dosing tanks form part of the operating unit of 3 acres of sprinkling filters. Each tank contains one 30-in. Miller siphon discharging through an independent pipe line to 1.5 acres of filters. The tanks as originally built each held 15 500 gal. of sewage and discharged once every 7 to 9 min. Sloping bottoms were built into the tanks after tests of the operation of the filter distribution system. The maximum rate of flow for maintaining intermittent operation is 7 440 000 gal. per 24 hours.

The sprinkling filters are built in units of 1.5 acres, each 473.5 ft. long and 140.5 ft. wide, with one unit on each side of a common pipe gallery. The depth of filter stone over the under-drains varies from 5.67 to 6.33 ft. The surface of each filter slopes away from the dosing tanks 0.8 ft. in 470. The under-drains are of the trough type covered with vitrified clay blocks and slope to a main collector in the pipe gallery at the rate of 0.67 ft. in 145. Except for a thin layer of larger stones over the under-drains, the filter stone is 1 to 2 in. in size. Each filter unit has 430 round-spray sprinkler nozzles of the Taylor type. The lateral distributors of 6 and 8-in. pipe are spaced 11.5 ft. on centers and the nozzles are 13.25 ft. apart on each lateral. The filter floor is of unreinforced concrete 5 in. thick, built in strips 10 ft. wide.

The effluent from the sprinkling filters flows through a main outfall conduit to the final settling tank. The first construction included only one final settling tank out of four proposed for the ultimate capacity. This tank is circular and has a liquid depth of 11.19 ft. at the center. The sewage is discharged at the center, 9.11 ft. above the bottom of the tank, and flows radially to weirs at the circumference. The sludge is removed by a Dorr mechanism to sludge pumps of the Barnes type set in an adjacent building. These pumps have a suction lift of about 11 ft., a capacity of 59 gal. per min., and operate satisfactorily. The settling tank is built of reinforced concrete with a bottom 48 in. thick to compensate upward pressure.

At a sewage flow of 8 000 000 gal. per day, the rating of the final settling tank is 1 810 gal. per sq. ft. of tank surface per 24 hours, or a displacement period of 37 min. Sludge is pumped through a wood stave force main over the dike to bottom-land along the river.

The office and laboratory building is about 30 ft. square and contains two floors and a basement. The upper floor is intended for living quarters for workmen, but has never been furnished for use. The basement contains the heating plant, a shop, and space for the sludge locomotive. The intermediate floor is partitioned into four rooms, two of which are fully equipped laboratories, one is the office, and the fourth, a drafting-room. The space appears to be ample. The building is of common brick with asphalt shingles.

The water supply for the plant comes from an infiltration gallery approximately 6 ft. in diameter and delivering by gravity through a 4-in. supply main about 300 ft. long to a centrifugal pump in the basement of the office building. The water is pumped into an elevated tank of 50 000 gal. capacity and is distributed about the grounds.

Trains of six sludge cars are hauled by a Plymouth gasoline locomotive to a sludge dump along the west dike. Some sludge has been pumped wet to adjacent cornfields.

The grounds about the plant are under-drained, but very little landscaping has been undertaken as yet. There is a by-pass sewer for the entire plant and one for the sprinkling filters.

OPERATION OF SEWAGE TREATMENT WORKS, 1924-1927

Quality of Sewage.—The quantities and characteristics of Decatur sewage are shown in Table 12 which gives the average monthly data. The chemical data are determined according to the Standard Methods of Water Analysis (American Public Health Association, 1923) on daily samples composited hourly and kept at 10 to 20° cent. in a water bath. The data are discussed under three divisions, A, B, and C, as follows:

Division A is the data from August, 1924, through June, 1925. During this period 6 000 000 to 8 000 000 gal. of relatively pure condenser water from the glucose refinery were discharging into the City sewers, thereby increasing the total dry-weather volume of sewage to 15 000 000 gal. per day, or more. Of this quantity, 6 500 000 gal. per day was by-passed around the plant, 8 500 000 gal. per day was passed through the Imhoff tanks, and all but 3 000 000 to 4 000 000 gal. per day was again by-passed around the filters. The data show the sewage of this period to be one of normal suspended matter, high in total nitrogen, oxygen consumed, and bio-chemical oxygen demand, but only one-half as strong as that received during the period discussed under Division B.

Division B is the data from July, 1925, through August, 1926. During this period the refinery condenser water was not entering the sewers, but was being returned to the impounding reservoir. This period was characterized by a normal rainfall and a steady increase in the corn grind from month to month. However, the efficiency of operation of the starch plant was improved after November, 1925, so that the strength of the sewage at higher corn grinds was less than it had previously been at a much lower grind.

TABLE 12.—ANALYSES OF CRUDE SEWAGE—MONTHLY AVERAGES.

Month.	Total sewage flow, in million gallons daily.	Suspended matter, in parts per million.	Oxygen consumed, in parts per million.	Total nitrogen, in parts per million.	5-day bio-chemical oxygen demand, in parts per million.	Total population equivalent.	Remarks.
1924:							
August	(15.0)*	216	117	34	350	256 000	Division A, 6 000 000 to 8 000 000 gal. per day, condenser water, diluting sewage.
September	(15.0)	246	173	41	415	305 000	
October	(15.0)	212	165	29	345	253 000	
November	(15.0)	218	184	30	370	272 000	
December	(15.0)	197	231	39	485	356 000	
1925:							
January	(15.0)	132	224	45	450	330 000	
February	(15.0)	227	228	45	455	334 000	
March	(15.0)	252	194	33	370	272 000	
April	(15.0)	232	171	26	310	228 000	
May	(15.0)	262	206	30	435	320 000	
June	(14.20)	248	193	40	498	366 000	
July	9.25	209	149	31	405	204 000	Division B, condenser water diverted.
August	10.80	298	162	30	503	216 000	
September	11.20	332	236	51	560	257 000	
October	10.45	334	352	60	710	365 000	
November	9.74	360	421	74	785	374 000	
December	9.53	291	380	77	740	346 000	
1926:							
January	9.88	324	418	69	748	362 000	
February	11.38	322	368	56	735	410 000	
March	11.80	305	343	52	663	383 000	
April	(13.50)*	278	299	42	528	349 000	
May	(12 50)	262	244	58	565	346 000	
June	(11.50)	222	172	42	384	216 000	
July	9.70	218	206	50	523	249 000	
August	11.50	235	199	52	584	309 000	
September	(20.40)*	232	168	35	325	325 000	Division C, flood-water dilution.
October	(15.60)	214	185	38	425	325 000	
November	(12.60)	285	236	35	520	321 000	
December	11.45	290	260	52	570	320 000	
1927:							
January	11.30	269	265	52	580	321 000	
February	13.40	229	240	47	498	327 000	
March	16.10	252	214	41	413	326 000	

* Calculated total sewage, including by-pass.

Division C is the data for eight months of abnormally high rainfall and river stage, with dilution of the sewage by storm water and infiltration. This period also represents further increases in efficiency at the starch works, as indicated by the decrease in population equivalent. The general trend of the chemical quantities was downward, due partly to dilution by infiltration and storm water, and partly to greater efficiencies at the starch works.

Operating Details.—The sewage passes first through a bar screen with openings between the bars of $1\frac{9}{16}$ in. From June 1, 1924, to September 1, 1926, a total of 110 cu. yd. of screenings was removed. This represents 0.0153 cu. yd., or 0.4 cu. ft., of screenings per million gallons of sewage. The screenings are buried daily.

The grit chambers are operated alternately and the velocity of flow is entirely controlled by the orifice weir at the outlet. The character of grit usually obtained is mostly sand, cinders, and fine coal. At certain periods of the year there is a tendency for paunch manure from two packing houses to settle toward the outlet ends of the chambers. This material is obnoxious to handle, but dries rapidly and causes no odor where used as fill.

The total of 541.5 cu. yd. of grit, or 2.16 cu. ft. per million gallons of sewage, has been obtained in 24 months of operation. The grit has been used for filling in low places and for replacing sand in the sludge drying beds. It has not been entirely satisfactory for this latter purpose because of its low density, which allows it to adhere to the dry sludge and be removed too rapidly from the beds. The possibility of using a sand-washing machine for reclaiming the heavier sand from the drying beds is being considered.

The Imhoff tanks have operated most satisfactorily considering that they have been run most of the time at 35% above their designed capacity. The resulting larger volume, due to the increased industrial sewage and the lack of sludge bed capacity, has caused the sludge to reach the slots of the flowing-through compartments each winter in January, thus reducing the efficiency of the tanks during January, February, and March. Fig. 9 shows the sewage flow and the percentage-removal of suspended matter in the Imhoff tanks to date (1927). The removal of bio-chemical oxygen demand has been between 20 and 22 per cent. This low result is attained because of the large amount of colloidal material in the starch works sewage.

Sewage was started through the Imhoff tanks on May 15, 1924, and vigorous foaming began on July 24, 1924. The foam rose over the free-boards of the gas vents and covered the flowing-through compartment to a depth of about 1 ft. However, well-designed scum-boards held the foam back so that very little of it escaped in the effluent of the tanks. Stirring and hosing the tanks added to the troubles at least temporarily. After about six weeks, the foaming stopped and was not again experienced until after the three-month shut-down in the summer of 1926 for the installation of the gas collectors. During the shut-down the sludge remaining in the tanks practically stopped gassing and apparently did not properly seed the fresh sludge when the tanks were again started in July. Numerous laboratory tests have shown that old sludge may become dead, so to speak, in so far as its inoculating capacity is concerned. Beginning in the fall of 1926, some of the tanks foamed mildly for three months, but in December the foaming became so violent that many of the gas-collecting domes were disconnected to prevent foam from entering the gas-piping system. It became necessary to remove the wooden sludge screen in each gas collector, because the "sludge-scum" which accompanied the foaming was very slimy and clogged the openings in the screen, thus creating considerable pressure. On releasing the screens some were blown from 4 to 6 ft. into the air, and the workmen got a foam shower bath.

Analyses of the gas during foaming show a high carbon dioxide content. The pH of the sludge liquor, at this time, was about 6.4, thus indicating that the acidity needed correction. To obtain some practical information on these foaming troubles several experiments were made.

One tank was not treated in any way; that is, foaming was allowed to take its own course. This tank foamed continuously for two months.

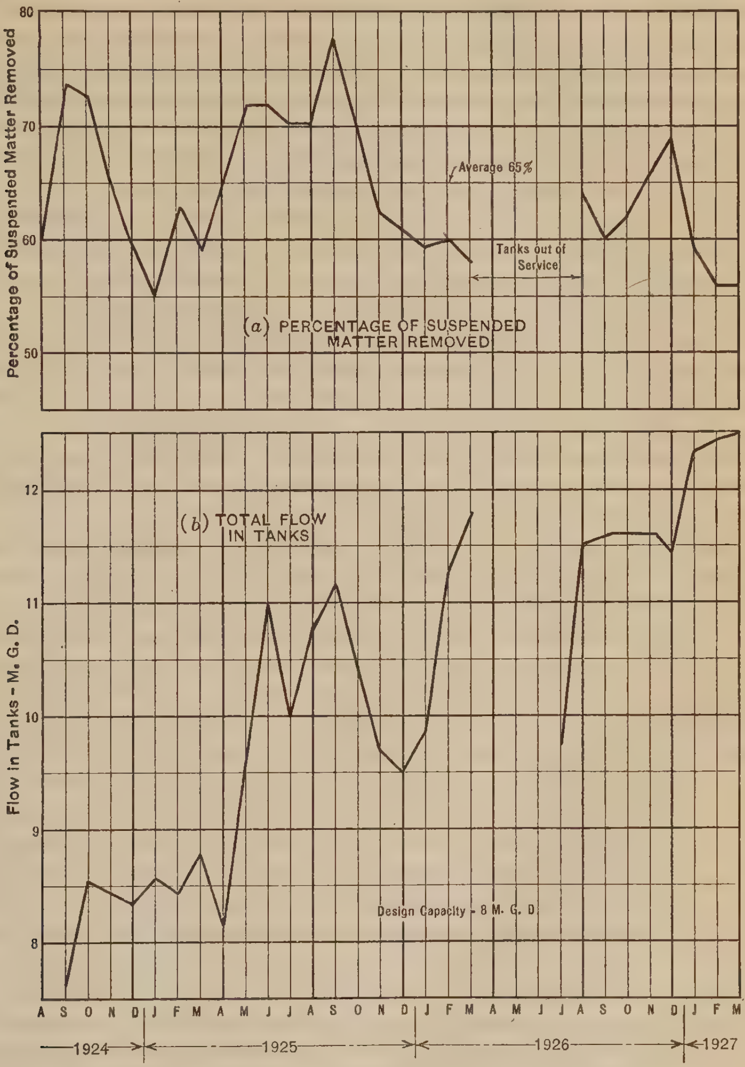

FIG. 9.—RESULTS OF OPERATION OF IMHOFF TANKS.

In one tank the depth of sludge liquor was measured from the surface to the solid sludge layer in the bottom. The acidity of the sludge liquor was titrated with Brom-thymol blue to a pH of 7.0, and the necessary quantity of lime calculated. The lime was added to each hopper down the gas-collector chimneys, through a chute, with a stream of water. This chute delivered the lime-water mixture about 4 ft. below the surface of the scum. Within 24 hours foaming had stopped, and the gas-collecting system was reconnected and placed in operation.

In another tank the lime was placed on top of the "foam-scum" in the flowing-through compartment. The influent of the tank was shut off and the scum hosed down. Foaming was greatly reduced, although not so effectively as where the lime was introduced directly into the sludge compartment.

In other tanks small quantities of lime were added to the gas vents every few days over a period of 2 to 4 weeks. More lime, more trouble, and more time were necessary to accomplish results in this way than by adding the lime carefully and slowly, but all in one day, as described previously.

Foaming that is not vigorous can be greatly reduced by removing the sludge scum from the gas vents and throwing it into the flowing-through compartments, or, better, by putting it on the drying beds. Regular stirring with water pressure is also helpful in the prevention of foaming.

The foregoing data are based on one period of experience with foaming. The same experience might not occur again, and several years of operation would be required to permit general conclusions.

No record has been kept of the volume of wet digested sludge removed from the Imhoff tanks, but 7 493 cu. yd. of air-dried sludge have been removed from the sludge drying beds. This is equivalent to 1.04 cu. yd. per million gallons of sewage treated.

Because of the high temperatures of the sewage due to the starch waste (70° Fahr. in winter to 104° Fahr. in summer), sludge digestion is very rapid and satisfactory all year around, even if the normal pH is as low as 6.8. At no time has there been any sludge drawn with an obnoxious odor, even when by mistake one tank was completely emptied of its sludge. Laboratory experiments with Decatur conditions have shown that at least 50% of the daily gas produced is given off from freshly deposited sludge in the first 24 hours. If the gas rate from the tanks was 100 000 cu. ft. per day and the tanks were by-passed for 24 hours, the rate would be reduced to 50 000 cu. ft. per day. This rapidity of decomposition probably explains why no foul-smelling gray sludge has ever been obtained, even by collecting a sample at the sludge line in the digestion chamber. It should be noted, however, that sludge drawn when serious foaming was taking place did not dry as rapidly as properly digested sludge, although there was present the usual "tarry odor".

The method of operating the sludge beds has been first to rake the sand smooth and then to run on from 12 to 15 in. of sludge. This sludge will dry in 10 days of good drying weather, and it is then forked into cars and dumped on low ground as fill. Considerable sludge can be taken from the beds during freezing weather. The cakes of sludge freeze to such a consistency that they are almost as easily forked as summer sludge. The sludge-bed capacity has recently been increased 41%, which will relieve the Imhoff digestion chamber of its heavy winter and spring load.

The sprinkling filter operation has been intermittent because its elevation is such that it is not operated at times of high water in the river, and because the plant was by-passed for three months in 1926 for the construction of gas collectors and for several months in 1927 for the construction of the pre-aeration plant. Since the filters were placed in operation they have operated about 50% of the time. Table 13 gives data on a typical run from May, 1925, until January, 1926. It shows the effects of the concentration of the sewage on the filter rates, loadings, and efficiencies, particularly when high concentration and cold weather coincide. The sprinkling filter rates ranged from 1 500 000 to 750 000 gal. per day as the bio-chemical oxygen demand of the sewage ranged from 500 to 750 parts per million and the sewage temperature from 90° to 70° Fahr. At these rates the total filtering capacity of the 3 acres was from 2 300 000 to 4 500 000 gal. per day, or about one-third that necessary for complete treatment of the entire sewage. A rate of 1 000 000 gal. per day, with a bio-chemical oxygen demand of the sewage of 500 parts per million is equivalent to a bio-chemical oxygen demand loading of 4 165 lb. per acre per day.

Some trouble with ponding has been experienced. This seems to be due to growths on the surface stones and is broken up by forking, harrowing, and with water pressure from a fire hose.

TABLE 13.—SPRINKLING FILTER DATA.

Month.	Rate of application, in million gallons per day.	5-day bio-chemical oxygen demand, influent, in parts per million.	Bio-chemical oxygen demand, loading, in pounds per acre per day.	5-DAY BIO-CHEMICAL OXYGEN DEMAND, IN PARTS PER MILLION.		Total nitrogen, raw sewage, in parts per million.	Total nitrogen, in pounds per acre per day.	Nitrates and nitrites, effluent, in parts per million.	Percentage of stability of effluent to methylene blue.
				Sprinkling filter effluent.	Secondary tank effluent.				
1925:									
May	1.7	371	5 250	25	....	30	425	11.3	90
June	1.76	388	5 700	24	....	40	587	7.2	90
July	2.0	290	4 830	26	....	31	519	4.3	90
August	1.5	344	4 300	32	20	30	375	7.4	90
September	1.28	405	4 320	25	10	51	545	9.2	90
October	1.21	530	5 340	79	50	60	606	5.5	60
November	0.74	610	8 760	130	70	74	457	6.5	40
December	0.79	595	8 920	71	30	77	455	17.0	60
1926:									
January	1.06	650	5 740	63	30	69	611	20.0	90

The data in Table 13 show the effective removal (by the secondary tank) of bio-chemical oxygen demand from the sprinkling filter effluent. This bio-chemical oxygen demand appears to come largely from materials sloughing off the filter stones. The suspended matter removal is about 66 per cent.

Odors.—Very strong decomposition odors were noticeable at the inlet to the screen chamber when the plant was placed in operation. These odors were due to the relatively high temperature, which caused decomposition of the very strong sewage in the lateral and intercepting sewers. A galvanized iron railing around the screen and grit chamber was turned snow white by zinc sulfide frost. During the first few months of operation, a flap-gate was made which held these gases in the interceptor. This greatly reduced the odor during the first two years of operation.

After the diluting condenser water from the glucose plant was taken out of the sewerage system, the odors about the plant were greatly intensified, particularly at all points of turbulent flow. Analyses of the air about the plant for hydrogen sulfide showed the intensity of odor to exist in about the following order: (*a*) Gas from Imhoff tank gas vents; (*b*) above the grit-chamber effluent near the outlet weir; (*c*) gas from the intercepting sewer; (*d*) the overflow by-pass of the Imhoff tanks; (*e*) the dosing tanks; (*f*) the screen chamber; and (*g*) leeward of sprinkling filters during spray. These results showed the intensity of odor, but did not show the quantity. For example, the total hydrogen sulfide liberated from the 3 acres of sprinkling filters must have been very high, but the dilution with air over so great a surface makes the hydrogen sulfide concentration quite low.

The high hydrogen sulfide content of the Imhoff digestion gas was a surprise. Routine analyses of the Imhoff gas over a period of six months revealed that the hydrogen sulfide content is proportional to the temperature of digestion. Fig. 10 gives the data on temperature of digestion, percentage of hydrogen sulfide present, and the computed total pounds of hydrogen sulfide produced per day from the total Imhoff tank digestion. These data show the large amount (100 lb. per day) of hydrogen sulfide liberated at summer temperatures of digestion.

It has only been possible to obtain quantitative data on the Imhoff gas, but after considerable study of conditions a qualitative estimate indicated that, roughly, 25% of the odors came from the grit chamber, 50% from the Imhoff gas vents, conduits, and by-passes, and probably 25% from the dosing tank and sprinkling filters.

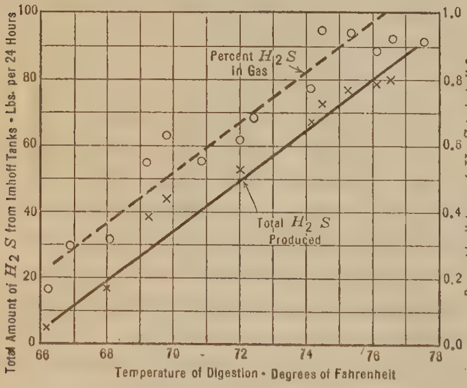

FIG. 10.—HYDROGEN SULFIDE IN IMHOFF TANK GASES, SEWAGE DISPOSAL WORKS, DECATUR, ILL.

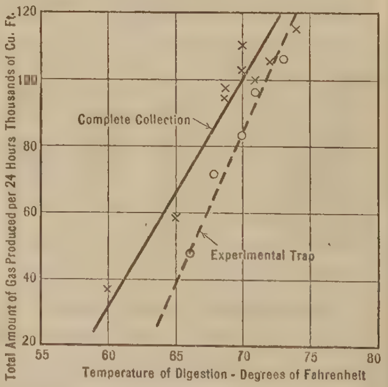

FIG. 11.—VOLUME OF GAS FROM IMHOFF TANKS, SEWAGE DISPOSAL WORKS, DECATUR, ILL.

The odors were reduced by covering the screen chamber, the grit chamber, the Imhoff tank conduits, and by-passes, and collecting all the Imhoff tank digestion gas. The chambers, conduits, and by-passes were covered with a 4 to 6-in. reinforced concrete slab containing suitable manholes and removable covers for accessibility. These covered sections were connected by 8 and 10-in. suction piping to an exhaust fan which forces the gases into a combustion oven. The details for the collection of digestion gases are given in Fig. 4. The gas is piped to the office building for heating and laboratory use and to the combustion oven for burning, thus destroying the odors of the digestion gas and also the housed-in odors.

Gas Production.—Before building the gas collectors the central gas vent of one tank was experimentally trapped by building a sloping wooden structure under the surface of the sewage, somewhat resembling the Imhoff collector.* This structure collected about one-twelfth the entire gas production. The gas from this collector was used in the laboratory and to heat the office and laboratory building during the winters of 1925 and 1926. The data regularly collected on this experimentally trapped gas were temperature of sludge digestion, volume of gas, hydrogen sulfide, calorific value, and chemical constituents. The relations of temperature to hydrogen sulfide and to total volume are shown in Figs. 10 and 11. The gas has an average calorific value of about 700 B.t.u. per cu. ft. The chemical composition varies as shown in Table 14.

TABLE 14.—CHEMICAL COMPOSITION OF GASES.

Gas.	Percentage by Volume.			
	Maximum.	Minimum.	Average.	With foaming tanks.
Methane	74	55	68	60–64
Carbon dioxide	34	16	22	30–33
Hydrogen	4	0	2	2
Nitrogen	10	4	6	7
Oxygen	0	0	0	0

The total volume of gas from the complete collecting system has been from 80 000 to 125 000 cu. ft. per 24 hours, except when the starch works was shut down or when melting snow and cold rains lowered the temperature and, by diluting the sewage, decreased the food supply for bacterial decomposition. At such times the gas volume has reduced to about 30 000 cu. ft.

The gas and odor-collecting systems have completely destroyed the decomposition odors from the screen and grit chambers, and from the Imhoff tanks and by-passes. There is a slight sulfur dioxide odor from the burned gases, but it is not objectionable even near the oven. The sprinkling filters still give off odors when in operation which leaves the last 25% of the odors to be eliminated or reduced by the new pre-aeration plant and by recovery of waste products at the starch works. The sprinkling filters were not in operation during the summer of 1927, because of high water and the construction work for the new pre-aeration plant. During this period there have been no odors at all from the other parts of the plant, thus showing its efficiency for collecting and burning the gases.

Utilization of the gas for generation of power for the pre-aeration plant has been considered but, due to the uncertainty of the future volume and strength of the starch works sewage, it did not seem advisable to use it at present. From the analyses, about 14 cu. ft. should develop 1 h.p.-hr. in a combustion engine. The gas is to be used for heating the air for the new pre-aeration plant before compression, and is now used (1927) for heating the building, for hot water, and in the laboratory.

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* *Engineering News-Record,* Vol. 91, No. 13, p. 512 (1925).

Labor Force and Annual Budget.—The labor force required to operate the plant has been an assistant chemist, a foreman, two full-time operators, a night watchman, a gardener to take care of the grounds and assist in other work, and two extra men during the sludge-drying season. The budget for the fiscal year 1926-27 for operation and maintenance of the plant is as follows:

Item.	Amount.
Supervision	$4 000
Labor payroll	10 000
Equipment, laboratory and engineering	600
Materials and supplies	1 120
Electric power, pumping station and plant	2 000
Gasoline, oil, and grease	300
Recording gauge	150
Landscaping	1 000
Testing station operation	2 000
Repairs to roadways	200
Contingencies	630
Total	$22 000

It is too soon to predict the cost of operating the pre-aeration plant. However, it will probably be necessary to add only one more operator to the staff. The power costs for pre-aeration have been estimated at $15 000 per year.

Laboratory Routine.—Rather complete analytical data have been kept on the raw sewage and effluents, particularly because of the unusual strength of the sewage and the influence of adjustments by the starch works in its plant process or otherwise.

During 1924 and 1925 the samples were analyzed daily, but beginning in January, 1926, bio-chemical oxygen demand and nitrite-nitrate nitrogen only were determined daily; the other determinations were run on samples made by compositing 2-day samples. This method gave results which are satisfactory and allowed more time for important research work.

The regular routine analyses made were: (*a*) Five-day bio-chemical oxygen demand, daily on raw sewage, experiment station samples, Imhoff effluent, sprinkling filter effluent, and secondary tank effluent; (*b*) suspended matter, settleable matter, permanganate oxygen consumed, total nitrogen, and ammonia were determined on the 2-day composite; and (*c*) nitrite-nitrate nitrogen, methylene blue stability, and dissolved oxygen were determined daily on aeration testing station and sprinkling filter effluents.

All analyses are made according to "Standard Methods of Water Analysis", (American Public Health Association, 1923), except that total nitrogen was determined instead of first boiling off the ammonia and then measuring the organic nitrogen. Ammonia is determined by direct Nesslerization.

The dilution water used for the bio-chemical oxygen demand determination was from the infiltration gallery which supplies the sewage plant water system. Dr. E. R. Greenfield* has described the importance of salts in dilution water and their effect on the 10 and 20-day oxygen demands. Dr. F. W. Mohlman recommends 0.5% $NaHCO_3$ in distilled water. The writers favor the establishment of a definite salt-containing dilution water for the bio-chemical oxygen demand determination. The composition of the tap water which has been used after aeration and standing 20 days is given in Table 15.

TABLE 15.—Composition of Tap Water.
($pH = 8.7$).

Item.	Parts per million.	Item.	Parts per million.
Calcium (Ca)	114	Carbonate (CO_3)	178
Magnesium (Mg)	57	Bicarbonate (HCO_3)	0
Sodium and potassium (Na and K)	12	Sulfate (SO_4)	112
Iron (Fe)	0	Chloride (Cl)	41
Ammonium (NH_4)	0.01	Nitrate (NO_3)	15

Oxygen consumed from permanganate bears a very definite relation to bio-chemical oxygen demand for each sampling point, for the condition of the sample, and for the period of year. At Decatur the oxygen consumed is used as a convenient yardstick from which is predicted the probable 5-day demand on the first day.

The present Imhoff installation may be relied on to give a 65 to 75% removal of suspended matter and a 22% removal of bio-chemical oxygen demand when treating as much as 11 000 000 gal. per day. This is the maximum volume which it will handle satisfactorily.

The additional sludge bed area, making a total of 1.37 acres (equivalent to 1.0 sq. ft. per capita based on 60 000 population), will give ample sludge capacity in the Imhoff digestion chamber for the sludge from the mixed sewage. There may not be sufficient sludge capacity to take care of the waste activated sludge from the pre-aeration plant which is to be returned for digestion into the Imhoff tanks, owing largely to the uncertainty as to its volume and temperature. Separate sludge digestion tanks may have to be added later.

The sprinkling filter capacities have varied from 700 000 to 2 000 000 gal. per acre per day and from about 4 000 to 5 700 lb. of 5-day bio-chemical oxygen demand per acre per 24 hours. For an average applied sewage strength of 500 bio-chemical oxygen demand and moderate summer temperatures, the sprinkling filter will handle about 1 200 000 gal. per day and 5 000 lb. of 5-day oxygen demand per acre 6 ft. deep. During cold weather the filter rate must be cut to somewhat more than 700 000 gal. per day, in order to keep the filter in condition. The maximum nitrogen loading with Decatur sewage

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* *Industrial Engineering Chemistry,* No. 18, p. 1276 (1926).

TABLE 16.—RECORDS OF OPERATION OF TESTING STATION.

Period.	Month and year.	Percentage of sludge return.	Percentage of sludge wasted to sewer.	Simplex aeration, in hours.	Re-aeration of sludge, in hours.	Cubic feet of air per gallon of sewage.	Displacement period, in hours, settling tank.	Filter rate, in million gallons per day.	Bio-Chemical Oxygen Demand,* 5 Days, 20° Cent.				Percentage of Removal, Bio-Chemical Oxygen Demand.†			
									Crude sewage.	Imhoff tank effluent.	Aeration tank effluent.	Sprinkling filter effluent.	Imhoff tanks.	Aeration.	Filter.	Total.
1	2/25	6.0	0	5.2	3.2	3.4	1.0	1.24	(397)	(318)	(190)	(52)	....	40	....	....
2	3/25	25.0	12	4.3	2.8	2.1	1.1	2.51	(305)	(244)	(104)	(24)	....	57	....	....
3	4/25	22.2	0	7.2	5.8	5.6	1.6	3.00	(303)	(243)	(152)	(28)	20	38	....	92
4	4/25	16.3	1	3.5	3.1	3.26	0.8	3.05	(352)	(282)	(148)	(28)	....	48	....	....
5	5/25	36.0	15.8	3.1	1.7	1.94	....	3.05	(450)	(360)	(130)	(28)	....	61	....	....
6	5/25	21.0	13.7	3.4	....	....	2.1	3.00	457	371	120	37	19	65	71	92
7	6/25	34.0	25.0	4.1	....	....	Varying	3.10	487	408	185	48	16	55	74	90
8	6/25	25.0	20.8	4.5	....	....	3.0	3.10	488	366	128	58	15	65	55	88
9	7/25	34.0	5.2	6.0	....	....	3.7	3.20	374	256	103	25	31	60	76	93
10	8/25	10.0	14.8	7.0	....	....	4.1	3.30	495	340	122	37	31	64	70	93
11	9/25	25.0	(?)	3.3	....	....	1.8	3.5‡	452	328	146	31	28	56	79	93
12	9/25	13.7	0.3	3.3	....	....	1.6	3.78	543	405	195	44	26	52	77	92
13	10/25	13.7	1.9	3.3	3.4	2.3	1.6	3.5§	761	551	237	38	28	58	84	95
14	10/25	13.7	(?)0.9	3.3	....	....	1.6	3.36	745	560	269	52	25	52	81	93
15	11/25	10.0	1.1	2.5	....	....	1.1	3.36	787	640	387	67	19	39	83	91
16	12/25	10.0	0.9	2.5	....	....	1.1	1.75	790	608	317	69	23	48	78	91
17	12/25	10.0	1.0	2.5	....	....	1.1	2.52	785	633	430	72	19	32	83	91
18	1/26	11.0	0.9	3.4	....	....	1.6	2.52	760	645	425	103	26	34	76	86
19	1/26	13.5	(?)	3.4	....	....	1.6	2.37	770	607	340	58	21	44	83	92
20	2/26	11.0	1.9	3.4	....	....	1.6	3.00	783	606	284	54	23	53	81	93
21	2/26	13.0	2.4	6.7	....	....	3.3	3.00 3.33 4.32 5.00	640	570	118	41	11	79	65	94
22	3/26	13.0	2.5	9.3	....	....	4.5 2.2	4.28	640	535	110	39	16	83	64	94
23	4/26	16.0	4.8	11.2	....	....	3.1	3.8	536	491	46	16	11	91	65	97

* Bio-chemical oxygen demand data in parentheses are calculated by factors from oxygen consumed and total nitrogen.

† Percentage removal, bio-chemical oxygen demand, is calculated on each step in the process separately and the last column contains the total percentage removal.

‡ Filter rate, 1.64, two days; 2.49, two days; 4.21, eight days; average, 3.5.

§ Filter rate, 3.78, eight days; 3.33, twelve days.

TABLE 16.—(*Continued*).

FINAL EFFLUENT.			TOTAL ORGANIC AND FREE NITROGEN, NH_3, BY KJELDAHL.				OXYGEN CONSUMED, $KMnO_4$, 30 MIN.				SUSPENDED MATTER, GOOCH.				SETTLEABLE MATTER.			
Relative Stability, Percentage.	Nitrate and Nitrite Nitrogen.																	
Sprinkling filter effluent.	Aeration tank effluent.	Sprinkling filter effluent.	Crude sewage.	Imhoff tank effluent.	Aeration tank effluent.	Sprinkling filter effluent.	Crude sewage.	Imhoff tank effluent.	Aeration tank effluent.	Sprinkling filter effluent.	Crude sewage.	Imhoff tank effluent.	Aeration tank effluent.	Sprinkling filter effluent.	Crude sewage.	Imhoff tank effluent.	Aeration tank effluent.	Sprinkling filter effluent.
69	....	1.6	47	37	34	....	277	153	95	53	216	92	79	55	2.2	0.4	0.3	0.8
90+	1.6	8.1	32	28	26	....	151	133	52	35	270	100	123	142	2.7	0.6	13.6	6.3
90+	0.3	6.7	28	28	29	....	150	111	76	44	215	77	197	130	3.6	0.4	16.9	2.7
90+	0.0	8.7	26	25	23	....	174	114	74	44	248	76	164	105	4.0	0.7	39.0	12.0
90+	0.1	6.8	32	28	21	....	210	138	65	43	269	75	131	93	4.2	0.7	93.0	37.0
70	0.4	5.0	32	28	26	9	210	108	85	38	251	78	48	82	4.0	0.6	77.7	2.8
80+	0.4	7.2	34	22	22	10	205	119	81	44	266	68	83	114	4.7	0.2	16.7	5.5
80+	0.1	5.8	42	38	28	15	191	102	88	56	244	73	107	99	4.0	0.5	39.0	14.5
90+	0.1	6.3	30	25	17	6	140	69	44	25	197	61	36	52	3.3	0.6	90.0	7.6
38	0.1	9.0	32	....	....	....	154	87	63	35	280	89	....	....	3.7	0.9	58.0	17.3
53+	0.3	5.9	45	....	38	22	199	125	67	39	322	61	50	28	5.8	0.8	0.3	0.5
85+	0.1	11.0	55	58	42	....	231	133	80	39	293	79	45	56	4.7	1.2	0.3	2.3
79+	0.2	7.6	60	60	43	....	375	196	96	36	354	95	88	36	4.6	0.9	0.2	1.2
90	1.3	8.9	69	63	49	....	439	249	116	57	367	127	99	86	4.8	....	0.1	3.3
54	2.0	6.6	79	76	62	....	396	254	125	54	356	124	77	59	5.0	....	....	1.8
49+	1.0	14.0	72	69	59	....	382	257	106	62	294	118	73	101	4.1	0.9	0.2	6.3
43	1.8	3.3	82	77	62	....	398	264	112	63	315	115	57	70	4.4	0.6	0.2	2.1
11	....	3.3	68	67	59	....	415	270	120	74	306	124	61	67	4.3	0.6	0.1	2.2
87	....	12.0	69	65	52	....	393	251	112	60	377	154	42	94	5.3	0.6	0.1	3.7
77	....	10.6	51	48	41	....	403	265	103	58	350	140	46	78	4.0	0.7	0.1	3.1
99	3.5	17.8 25.0 17.0 13.0	50	48	35	....	354	254	79	45	282	122	64	47	3.7	0.8	0.1	1.6
99	2.5	19.6	33	50	37	....	319	260	73	52	310	141	84	40	4.5	0.9	10.4	1.6
99	2.7	18.0	43	51	16	....	328	263	71	40	289	147	49	23	2.9	1.0	2.7	0.4

seems to be about 552 lb. per acre 6 ft. deep. The 3 acres of sprinkling filters have only one-third to one-fourth the capacity required to treat the total mixed sewage without pre-aeration.

Pre-Aeration Testing Station, 1925-1926

Purpose.—The purpose of the pre-aeration testing station was to determine the applicability of a combination of aeration and sprinkling filters to sewage treatment at Decatur. Experience with the operation of the main sewage treatment plant indicated that a relatively large area of sprinkling filters would be needed, amounting to about 9 to 12 acres, depending on the process adjustments at the starch works. Such an increase in filter area was beyond the means of the Sanitary District and was considered hazardous from an odor standpoint.

The installation at Birmingham, England, of sewage treatment works, comprising a short period of aeration of settled sewage, with increased loading of existing sprinkling filters, looked promising. A visit to Birmingham was made, therefore, by Paul Hansen, M. Am. Soc. C. E., in 1924 and, later, by one of the Trustees of the District. The construction of the testing station as described was then recommended and authorized. Actual operation began on February 23, 1925.

Operating Records.—A complete record of operation is given in Table 16. The influent to the testing station comprised settled sewage from the Imhoff tanks. The volume of this influent sewage, the sprinkling filter influent, the returned sludge, and the excess sludge wasted to the sewer were all measured in orifice boxes. The volume of air used for sludge re-aeration was measured by a displacement meter. The power for operating the Simplex aerator was measured by a recording watt-hour meter. The orifice boxes for measuring the influent sewage and the liquor going to the sprinkling filters were provided with constant level overflows so that the flow through the orifice was constant. The orifice boxes for the sludge did not have constant level overflows. However, by careful attention, a close record was kept of the sludge quantities despite the difficulties occasioned by the wide variation in the density of the sludge.

The sprinkling filter was first dosed from a tank the outlet of which was controlled by a cam which opened and closed a valve in the influent line to the filter. The operation of this method of control was satisfactory, but required numerous cams for changing the rate of flow and very careful adjustment of the sprinkling filter orifice box in order not to overflow the dosing tank. The cam, therefore, was displaced by a siphon which delivered sewage to the filter at the rate at which it was pumped to the siphon tank. The air vents in so small a siphon were occasionally plugged with slimy growths so that they had to be cleaned regularly. The sprinkling filter was made of stone from the large filters and, therefore, was seeded from the beginning of the experiments.

Operating Periods.—The testing station was operated practically continuously from February 23 to August 20, 1925, when the motor for the Simplex aerator burned out, occasioning a shut-down until September 11. During this time the operation was divided into ten periods, each period comprising a run of 12 to 25 days. During each of these periods the various operating elements, such as the rate of flow, the displacement period in the aeration tank, the quantity of sludge returned, and the rate on the sprinkling filters, were kept constant. During the first nine periods the sewage contained the diluting refinery condenser water which caused a weaker mixed sewage than was received later during the tests. The sewage during the tenth period was increased in strength by the removal of the condenser water and decreased by a relatively low corn grind. During these first six months of operation much was learned about the behavior of an under-aerated sewage and sludge, and this period may be designated as experimental.

Operation was resumed on September 11, 1925, and continued without any appreciable interruption until April 7, 1926. During this time there were thirteen additional operating periods each of 12 to 20 days' duration, during which the rates of flow, the aeration periods, the quantity of sludge returned, and the sprinkling filter rates were held constant; except that during Runs 11, 13, and 21, the sprinkling filter rates were varied as indicated in Table 17. The crude sewage during this period was typical of the normal strong mixed sewage without dilution by the refinery condenser water or unusual rainfall. Its strength was influenced by considerable fluctuations in the corn grind. The strongest sewage was received during the coldest weather when rates of oxidation were at a minimum and difficulties with frozen pipes somewhat interfered with continuous operation.

TABLE 17.—SPRINKLING FILTER LOADINGS.

Period No.	Bio-chemical oxygen demand, parts per million, raw sewage.	Hours, aeration.	Bio-chemical oxygen demand, parts per million, influent.	Filter rate, in million gallons per day.	Bio-chemical oxygen demand, pounds per acre per day.	Data on Sprinkling Filter Effluent.			Total Nitrogen.		Pounds Total Nitrogen per Acre per Day.	
						Bio-chemical oxygen demand, parts per million.	Nitrate, parts per million.	Stability, percentage.	Raw sewage.	Filter influent.	Raw sewage.	Filter influent.
11	452	3.3	146	3.5*	4 250	31	5.9	53.9—	45	38	1 320	1 110
12	543	3.3	195	3.78	6 150	44	11.0	85 —	55	42	1 730	1 330
13	761	3.8	237	3.5*	6 900	88	7.6	79 —	60	43	1 750	1 260
14	745	3.3	269	3.36	7 550	52	8.9	90	69	49	1 950	1 370
15	787	2.5	387	3.36	10 800	67	6 6	54	79	62	2 210	1 740
16	790	2.5	317	1.75	4 620	69	14.0	89 —	72	59	1 050	860
17	785	2.5	430	2.52	9 030	72	3.3	33	82	62	1 730	1 310
18	760	3.4	425	2.52	8 920	103	3.3	11	68	59	1 430	1 240
19	770	3.4	340	2.37	6 720	58	12.0	87	69	52	1 360	1 030
20	783	3.4	284	3.00	7 100	54	10.6	77	51	41	1 280	1 030
21	640	6.7	118	3-5	2950-4910	41	18.-13.	99	50	35	1250-2080	880-1460
22	640	9.3	110	4.28	3 920	39	19.6	99	53	37	1 900	1 320
23	536	11.2	46	3.3	1 260	16	18.0	99	43	16	1 190	440

* Average.

Pre-Aeration Sludge.—One of the major problems in the operation of the testing station was to keep the under-aerated sludge in a settleable condition.

The data in Table 16 giving settleable matter as cubic centimeters per liter, show that during the first ten periods of operation a considerable volume of sludge was discharged over the settling-tank weirs on to the sprinkling filter. The relatively light character of this settleable matter is indicated by the proportionately lower quantities of suspended matter. During these periods the sludge was very thin and often would not settle appreciably in 2 hours. This thin fluffy condition of the sludge was first attributed to a lack of sufficiently heavy solids in the settled influent to the aeration tank, and attempts were made to procure a thicker sludge by (*a*) returning a larger percentage of sludge to the aeration tank; (*b*) increasing the period of sludge re-aeration to 5 hours; and (*c*), increasing the aeration period to 7 hours. None of these methods was entirely satisfactory. Increasing the quantity of returned sludge made bulking worse; and increasing the time of sewage aeration and sludge re-aeration, within the limits used, had no appreciable effect. It was finally found that each period of aeration and its strength of sewage gave a characteristic sludge which could be maintained and settled if not more than about 10% by volume of sludge after settling 1 hour was kept in the aeration tank liquor.

Operating experience clearly indicated that sludge should not be allowed to remain or accumulate in the settling tank. The solution of the sludge-handling problem appears to be a prompt and regular drawing of sludge so as to maintain only relatively fresh sludge in circulation as shown during Periods 17, 18, 19, and 20, in Table 16. During Periods 17, 18, and 19, the sludge was black, flocculant, with a rather septic odor, and settled rapidly. During Period 19 a considerable quantity of sludge was lost because of frozen sludge pipes. Just before the start of Period 20 the sludge line broke and all sludge was lost so that during this period the operation of the aeration tank was with entirely fresh sludge. The removal of 5-day bio-chemical oxygen demand by the partial aeration during these periods was, as follows:

Period.	Percentage removal of 5-day bio-chemical oxygen demand.
17	32
18	34
19	44
20	53

Thus, with the fresher sludges the removal of bio-chemical oxygen demand was greater.

Settling Tanks.—With partial aeration, the condition of the sludge makes the capacity and rating of the settling tanks important. In the testing station, the sludge was satisfactorily settled with a displacement period of 1.1 hours and a surface rating of 1 050 gal. per sq. ft. per 24 hours. When severe bulking of the sludge took place, it could not be settled with the maximum available displacement period of 3.6 hours equivalent to a surface rating of 320 gal. per sq. ft. per 24 hours. With regular and sufficient removal of the excess, the sludge may be kept sufficiently fresh and active and in condition to permit satisfactory sedimentation. This active sludge is in no way a typical activated sludge from complete aeration works, except in the physical nature of the floc. The color is gray to black and not brown. The odor is disagreeable, but not septic. The operation of the testing station indicated that the volume of sludge in the aeration tanks should not be more than 10% by volume as determined by sedimentation in cylinders for 1 hour. If a poorly settled sludge is developed, the best procedure is to remove it and to start again building up a new sludge. This is often almost white, but becomes more gray in color each day.

Periods of Aeration.—The period of aeration has a very definite relation to the amount of purification as measured by the percentage removal of bio-chemical oxygen demand. Fig. 12 shows the reduction in the 5-day bio-chemical oxygen demand for different periods of aeration. Although the longest aeration period was 11.2 hours, the trend of the operating data indicated that about 16 hours might be required for complete treatment of the Decatur sewage after preliminary sedimentation in order to turn out an effluent containing sufficient nitrates to insure stability with maintenance of a satisfactory sludge.

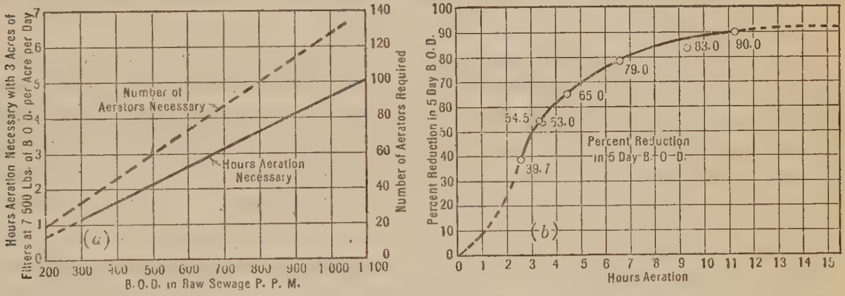

FIG. 12.—PRE-AERATION REDUCTION IN BIO-CHEMICAL OXYGEN DEMAND, SEWAGE DISPOSAL WORKS, DECATUR, ILL.

The maximum capacity of the testing station permitted only 6 hours re-aeration of the returned sludge. Under Decatur conditions this re-aeration did not materially improve the quality of the sludge. It was considered that a larger capacity in the pre-aeration tanks was preferable to provision for sludge re-aeration. In the full-sized plant connections have been made for the later construction of sludge re-aeration tanks should they be found desirable. The testing station operation showed that 6 hours of sludge re-aeration are of little value.

Operation of Sprinkling Filter.—The results of operating the small sprinkling filter are shown in Table 17. These data indicate that on a filter 6 ft. deep, about 7 500 lb. of 5-day bio-chemical oxygen demand can be satisfactorily handled per acre per 24 hours. It may be possible to handle a larger quantity than this at summer temperatures. This loading, under Decatur conditions, represents a dose at the rate of between 2 500 000 and 3 500 000 gal. per acre per 24 hours.

Table 16 gives the removal of 5-day bio-chemical oxygen demand by the several parts of the plant. From crude sewage to sprinkling filter effluent the removal of 5-day bio-chemical oxygen demand was, in general, somewhat more than 90 per cent. The testing station included no secondary or final settling tank for the sprinkling filter effluent. By such sedimentation, the bio-chemical demand of the sprinkling filter effluent may be further reduced from 20 to 50% of the final settling tank influent, thus increasing the over-all removal of bio-chemical oxygen demand to about 95 per cent.

Studies at the testing station showed that within reasonable limits, the sprinkling filters could be operated at a relatively constant loading of 5-day bio-chemical oxygen demand (in the partly aerated sewage) amounting to 7 500 lb. per acre per 24 hours. Thus, the quantity of effluent delivered to the sprinkling filter was proportionate to its strength and was fixed at about 22 500 lb. of bio-chemical oxygen demand for the 3 acres. It is possible, therefore, to draw a rough balance between the amount of pre-aeration necessary for any given strength of sewage with the 3 acres of sprinkling filters. Such a computation would include the over-all annual costs reflecting the fixed charge for construction and the annual cost of power and labor, so that the best economy for each condition might be secured. Fig. 12 gives the amount of aeration needed for different strengths of sewage calculated for dosing 3 acres of filters at the foregoing loads.

Odors from the aeration tank and sprinkling filter were not noticeable during the first ten operating periods, while the strength of the sewage was less than 500 parts per million of 5-day bio-chemical oxygen demand, but a very decided "animal den" odor was observed about the aeration tank as the strength of the sewage increased to more than 600 parts per million of 5-day bio-chemical oxygen demand. This odor may have been due to the stronger sewage, but was more probably occasioned by decreased aeration and a more septic sludge. With the process adjustments at the starch works the mixed sewage to be handled in the larger plant will have a 5-day bio-chemical oxygen demand of about 300 parts per million. With this weaker sewage, the new plant is not expected to give off any objectionable odors.

Basis of Design, New Pre-Aeration Plant.—The design of the new pre-aeration plant is based on a dry-weather sewage flow of 10 000 000 gal. per day and a maximum rate of 16 000 000 gal. per day, with additional capacity in the aeration tanks for 10% of the returned sludge. Analyses of the starch works sewage and the city sewage indicate that after the process adjustments at the starch works the mixed sewage will have a 5-day bio-chemical oxygen demand of somewhat more than 300 parts per million. Fig. 12 indicates that a sewage of this strength will require pre-aeration for about 1.25 hours to prepare it for application to the existing 3 acres of sprinkling filters. Because of possible irregularities in the starch works sewage and because of the retarding effect of cold and wet weather, it was decided to give the aeration tanks a displacement period exclusive of returned sludge of 2.75 hours, when the sewage flow was 10 000 000 gal. per day. This is equivalent to 1.56 hours at the higher rate of 16 000 000 gal. per day.

The settling tanks were also designed on a relatively conservative basis so that occasional sludge, difficult to settle, can be better recovered. Therefore, a displacement period of 2.6 hours, equivalent to a surface rating of 833 gal. per sq. ft. per 24 hours, was adopted. These capacities are about twice those found desirable during the test periods, 11 to 20 (Table 17).

PRE-AERATION PLANT, 1926-1927

General Arrangement.—After the testing station had been put in operation plans were prepared and contracts let for the construction of a pre-aeration plant with a rated capacity of 10 000 000 gal. per day. The plant comprises aeration units, settling tanks, blower house, and appurtenances; and brings the capacity of the sewage disposal works to about 150 000 equivalent population.

During the operation of the pre-aeration testing station, conferences were held with the officials of the starch works relative to process adjustments within the starch plant looking toward a reduction in the strength of the sewage. At that time the organic matter in the starch works sewage was roughly indicated, as follows:

Plant.	Pounds of total organic and free ammonia nitrogen in starch works sewage per bushel of grind.
Argo	0.06
Pekin	0.02
Decatur	0.14

The officials of the starch works at Decatur agreed to make process adjustments in their plant. At the time of the conferences in 1926, the equivalent population of the starch works sewage had, at times, exceeded 300 000. It was agreed that process adjustments would be made in the starch works so as to reduce the population equivalent to about 90 000 by July 1, 1927; and thereafter to a population equivalent of about 1 000 for each 1 000 bushels of corn ground and at a rate to keep pace more or less with the growth of the city. The Sanitary District agreed to install a pre-aeration plant to bring the capacity of the disposal works up to about 150 000 by July 1, 1927. Both agreements have been fulfilled except as to time, which was extended about four months. Thus, the Sanitary District in effect provides a somewhat liberal capacity for its future requirements and permits the starch works to utilize the available capacity for the future.

The pre-aeration plant has been built east of the present sprinkling filters, in the west end of the space allotted to future sprinkling filters in the original layout. The new blower house stands between the aeration tanks and the sprinkling filters. Along the east side of the new settling tanks, space is reserved for sludge re-aeration tanks should they be found desirable; and, in general, future extensions of the pre-aeration plant are planned in an easterly direction.

In order to provide head for the new units, it was not considered necessary to raise the elevation of the flow line in the Imhoff tanks, although this was possible because of the contingent allowance of 1.0 ft. in the original design and the relatively ample free-board of 23 in. The operating level of the dosing tanks however, was adjusted from Elevation 601.90 to Elevation 601.25, thus providing a head of 1.36 ft. for the operation of the pre-aeration plant. The hydraulic computations were based on a maximum rate of sewage flow of 16 000 000 gal. per day, an average of 10 000 000 and a minimum of 8 000 000 gal. per day.

Aeration Units.—There are six aeration units each 120 ft. long, 16 ft. wide, and 14.5 ft. liquid depth over the aeration plates, with 14 in. of freeboard. Including an allowance of 10% for returned sludge, the aeration tanks have a displacement period of 2.5 hours with a sewage flow of 10 000 000 gal. per day.

The aeration provides spiral flow from two rows of filtros plates along one side of each tank (the so-called Manchester type), the plate area amounting to 10.65% of the tank area.

An inlet conduit extends along the north end of the six tanks from which the sewage flows into the center of each tank through 24 by 24-in. sluicegates. Return sludge is pumped into the sewage at the inlet or west end of this conduit. The aerated sewage flows out of the tanks over weirs and then through Venturi meters to the settling tanks. There is one Venturi meter for each two aeration tanks so that the sewage flow can be distributed as desired. Indication of this flow is carried by air to the sludge pump houses. A pipe gallery extends along the south end of the aeration tanks, with a sludge pump house at the southeast end of the gallery housing the Venturi meter indicators, valves, piping, and sludge pumps, and connecting directly with the blower house at the west end.

There are two settling tanks each 77.5 ft. square, with a liquid depth of 10.37 ft. at the periphery and 13.69 ft. at the center. The total displacement period above the sloping bottom is 2.6 hours and the settling rate is 833 gal. per sq. ft. of tank surface per 24 hours, both for a sewage rate of 10 000 000 gal. per day.

The sewage will enter the tanks from a conduit along their east sides through six inlets for each tank. The effluent is discharged over adjustable weirs extending along the entire width of each tank and into a conduit connected to the dosing tank.

Each tank is equipped with a Dorr mechanism for the removal of sludge. The sludge passes through an 8-in., cast-iron pipe built under the tanks to an open well at the east end of the pipe gallery. Two 8-in. centrifugal pumps send the sludge either to the main conduit leading to the Imhoff tanks or to the conduit leading to the aeration tanks. There is a Venturi meter for each sludge line, and the division of flow is controlled by gate valves in the sludge lines.

The blower house is 50 ft. long by 26.5 ft. wide inside and has space for four blowers, a boiler for heating the air, and other appurtenances. There are two gear-connected General Electric blowers of the centrifugal type, operating at 11 850 rev. per min., each having a capacity of 3 500 cu. ft. of free air per min. At the average sewage flow of 10 000 000 gal. per day, this is an installed capacity equivalent to about 1.0 cu. ft. of air per gal. of sewage.

The characteristic curves of this type of blower are shown in Fig. 13. Curves *A*, *B*, and *C* are based on air temperatures of 35°, 50°, and 95° Fahr., respectively, and barometric pressures of 14.85, 14.4, and 14.0, respectively. With one blower in operation, the rate of air delivery can be varied from about 0.4 to 0.6 cu. ft. per gal. of sewage, with very little loss in efficiency.

The characteristic curves show that for equal volumes of free air, the warmer the air the less the power required. In view of the fact that considerable gas from the Imhoff tank was available, it was considered advisable to maintain the air going to the blowers at a temperature of about 95° Fahr. The air is drawn through louvres in the blower-house wall, through thermostatically controlled steam-heating coils, through a cell-type air filter, and thence to the blowers. The coils have a capacity to raise 3 500 cu. ft. of air per min., about 70 to 100° Fahr., depending on the temperature of the incoming air. The coils are supplied with steam from the blower-house steam-heating system. The cell filters have a capacity of 7 500 cu. ft. per min.

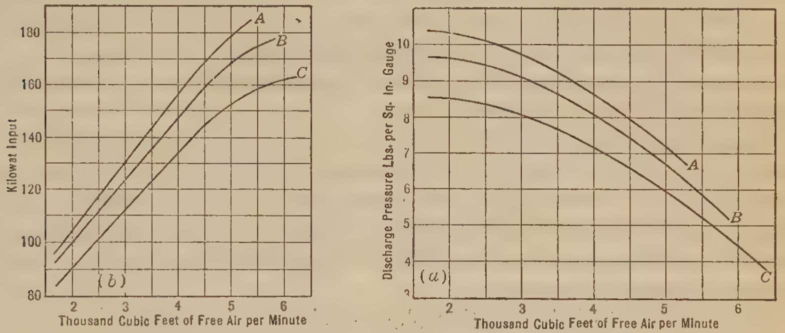

FIG. 13.—CHARACTERISTIC CURVES OF CENTRIFUGAL COMPRESSOR, SEWAGE DISPOSAL WORKS, DECATUR, ILL.

Air deliveries will be measured through a Bailey meter in the blower house on the main 18-in. cast-iron air line. This air line passes through the pipe gallery with 10-in. branches to each aeration tank and a 4-in. branch to all conduits. The rate of air flow to the three 10-in. and one 4-in. air lines is indicated in the sludge pump house by manometers which indicate the differential across the flow nozzles inserted in the several air lines. In this way, the flow in all the branches may be readily distributed by suitable adjustment of the valves. The air lines are provided with condensate traps for the removal of water. The blower house is built of common brick on a concrete foundation and has an asphalt shingle roof.

Excess sludge is pumped into the crude sewage near the bottom of the riser conduit at the northeast corner of the Imhoff tanks. The excess activated sludge is thus being digested with fresh sludge in the lower compartments. With the reduced strength of the starch works sewage, it is expected that there will be sufficient sludge digestion capacity in the present tanks to handle the excess activated sludge. Within a few years, however, it is planned to increase the sludge digestion capacity by adding separate sludge digestion tanks.

The operation of the testing station indicated the possibility of running the sprinkling filters at a rate of 3 000 000 gal. per day per acre. Therefore, it was necessary to increase the capacity of the dosing tanks, to install additional siphons, and to put in many more nozzles.

With the original installation of dosing tanks and 860 nozzles for the 3 acres of filters, the maximum rate of dosing was about 2 500 000 gal. per day per acre, or a total of 7 500 000 gal. per day, without putting the siphons into continuous operation. The nozzles were originally of the Taylor circular spray type with $\frac{15}{16}$-in. orifices spaced at 13 ft. 3 in., center to center. They were replaced with 1 312 Taylor circular spray nozzles with $\frac{3}{4}$-in. orifices spaced at 8 ft. 10$\frac{1}{4}$ in., center to center. The original system of distribution piping was used, and the additional nozzles were installed by means of branch pipes from the taps already made in the lateral distribution. The closer spacing and greater number of nozzles allowed a lowering of the high-water level in the dosing tank of 0.65 ft., which was made available for the pre-aeration plant.

The remodeled dosing tanks and filter distribution system are designed to handle an average flow of 10 000 000 gal. per day, with uniform distribution and can take care of nearly 16 000 000 gal. per day, without appreciable loss of uniformity and still maintain intermittent operation of the siphon. The cycle of operation at average flow is slightly more than 20 min., of which time the nozzles are in operation about 8 min.

During the operation of the testing station, odors were observed at the aeration tank. These were characterized as similar to the animal den odors at the "Zoo". The intensity of such odors from the large aerating tank and with the weaker sewage from the starch works has not yet been determined. However, provision has been made for housing the aerating tanks and for burning the gases if necessary.

Costs.—The cost of the aeration plant is given in Table 18.

TABLE 18.—COST OF AERATION PLANT.

Item.	Amount.
Contract 5	$142 785.71
Blowers	16 000.00
Switchboard	3 397.00
Oil filter equipment	496.00
Air meter	943.00
Air filter	295.00
Aeration plates	2 800.00
Dosing tank equipment	2 550.00
Additional piping	1 351.35
Blower house	44 663.00
Wrecking dosing tank	1 100.00
Total	$216 381.06

The unit prices of the major items were stated in the contract as follows:

Earth excavation	$0.50	per cu. yd.
Rock excavation	5.00	" " "
Reinforcing steel	0.053	per lb.
Concrete in floors and footings	12.25	per cu. yd.
Concrete in walk and conduits	20.00	" " "
Miscellaneous steel and wrought iron	0.13	per lb.
Miscellaneous iron castings	0.13	" "
Cast-iron pipe, bell and spigot	0.037	" "
Cast-iron specials, bell and spigot	0.093	" "
Cast-iron specials, flanged	0.132	" "
Subway grating	1.65	per sq. ft.

Conclusion

The project at Decatur up to the final stages of operation has required nearly fifteen years of investigation and construction effort. Portions of the work, such as the organizing of the Sanitary District, and the treatment of the starch works sewage, were unusual and required careful and gradual working out. The persistence of the effort is noteworthy and is due in large measure to the continued service of the Board of Trustees, one of whom has remained throughout the entire life of the Sanitary District. It is the hope of the writers that the foregoing record will be useful in the consideration of other sewage disposal problems.

DISCUSSION

GEORGE W. FULLER,* M. AM. SOC. C. E.—Among the novel features of the Decatur plant, pre-aeration perhaps comes first to mind as this is one of the first instances in America where that arrangement has been studied. It became a factor in respect to cutting down the load on the sprinkling filters. That plan of course has been studied in Europe in a few instances, particularly at Birmingham, England, where it has enabled the doubling of the load on the filters when it was found difficult to increase the available area or to get more money for construction. One hour is the period of pre-aeration at the Birmingham Plant and a portion of the excess sludge is aerated and conditioned in a tank. This provides about 16 to 18 hours of flow.

This pre-aeration may be applicable in places where ordinary sedimentation would be sufficient for perhaps twelve months in most years, but where occasionally there may be need, for a few months of very dry weather, of something beyond ordinary sedimentation. This idea relates to pre-aeration as a device for removing colloidal matter, not as a nitrate producer and not as a reducer of dissolved organic matters. There are plants, perhaps, like the one at Columbus, Ohio, at which advantage is taken of the diluting effect of stream flow available for a number of months of the year, and where during periods of low stream flow, there should be more treatment than is afforded by ordinary sedimentation.

Another arrangement serving the same general purpose is what Karl Imhoff, M. Am. Soc. C. E., calls contact aerators. These are used in half a dozen plants in the Ruhr District, in Germany, to get a better removal of the organic matter than is possible by sedimentation. After flowing through those tanks, the sewage goes along the stream bed a short distance to the Rhine. During this travel the mixed Ruhr water and treated sewage become well aerated. However, the colloids caused official complaint from across the National Boundary in Holland where the fishermen complained bitterly of fungus growths interfering with their nets.

Whether pre-aeration as described by the authors as a means of lessening the load on the sprinkling filters is a better arrangement than contact aerators, the speaker does not know. It certainly suggests possibilities for the solution of some types of problems.

The pioneering aspects of the Decatur plant are interesting as regards the collection and burning of gas to eliminate odors. That is an important step in advance. The observations in the paper as to lime also impress the speaker because of experiences recently with efforts to handle some of the complications which arise at intervals with Imhoff tanks, particularly at a plant that went in service in the fall of 1926 at Newcastle, Pa.

After some months of operation during the winter, when no effort was made to increase the temperature of the sewage or sludge, there was a tremendous amount of foaming followed by acid production in the spring. This acidity was not materially affected by the addition of small doses of lime. About ten years ago, at Plainfield, N. J., acid production became so pronounced that additions of lime were almost futile. When the pH-value becomes considerably less than 6, say, 5.2, the addition of lime in small quantities is not effective. The only recourse is to put in sufficient lime to bring the pH-value up to about 7, or more.

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* Cons. Engr. (Fuller & McClintock), New York, N. Y.

The question of handling gas at Newcastle has proved bothersome in that the arrangements for collecting it were improvised after the contract had been let, and they permitted the clogging of the gas screens. A further complication resulted from the large quantity of scum. While the gas collector is submerged, nevertheless its arrangement permits the space beneath the gas exits to fill with floating matter, so that the original facilities for cleaning it are now inadequate. Hence, at that plant there were three problems: (1) Inadequate screening provided by 2-in. clear openings; (2) inadequate protection of the gas exits which could be improved by the use of gas screens that do not clog so readily; and (3) inadequate arrangement for flushing the scum that accumulates in the **A**-space beneath the covers of ordinary gas vents, which covers have a longitudinal slope to lead the gas to the several screened exits.

JOHN F. SKINNER,* M. AM. SOC. C. E.—This paper is very interesting and the speaker wishes to add a few words on one point that is based on recent experience in Rochester, N. Y. A section of that city is supplied with water from Lake Ontario by the Rochester and Lake Ontario Water Company. This water is coagulated, passed through pressure filters, and chlorinated.

The Water Company's intake is 1.5 miles west of the mouth of the Genesee River and an occasional east wind carries to the intake a current of river water polluted with sewage and industrial wastes. The plant of the Rochester Gas and Electric Corporation has been in the habit of discharging into the river, 8 miles from its mouth, an average of 20 000 gal. per day of ammonia-still waste containing about 350 lb. of phenol ($C_6 H_5 OH$). Whenever the polluted river water reached the intake, the chlorine dose was increased, and complaints of the taste were numerous.

It was thought that if this gas-house waste could be discharged into the city's intercepting sewer, passed through the main disposal plant (consisting of Imhoff tanks), and then discharged with the effluent into Lake Ontario, 7 000 ft. from shore, in water 50 ft. deep and 2 miles farther from the water-works intake than the mouth of the river, greatly increased dispersion would take place, and if the waste reached the water-works intake it would be greatly diluted.

In order to determine whether this waste could be safely handled in the tanks at the main plant without upsetting their action and their ability to digest sludge properly, a large-scale experiment was conducted at the City's small "Charlotte" Plant† where the discharge averaged 500 000 gal. daily. The Imhoff tank has an inside diameter of 38 ft. In this tank the sewage flow is semi-circumferential, and a steel dome was placed over the central opening and its lower edge sealed by setting the dome in an annular groove of concrete on the deck and filling this groove with water.*

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* San. Engr., City of Rochester, Rochester, N. Y.

† *Transactions,* Am. Soc. C. E., Vol. 91 (December, 1927), p. 524.

The experiment was begun on June 3, and ended on September 2, 1927. Physical and chemical constants were first determined from four samples of normal sludge and later these were compared with fourteen samples of sludge after the continual addition of ammonia-still waste. The ratio was 1 gal. of waste to 1 000 gal. of sewage, or 500 gal. of waste daily, containing about 8.75 lb. of phenol, to 4 166 000 lb. of sewage, or 2.1 parts of phenol per 1 000 000 parts of sewage by weight.

The conclusions from the experiment were that when phenol wastes are added to sanitary sewage at a concentration of 2 parts per million of phenol, no harmful effects result. The operation of the Imhoff tank can be carried on as usual and the digestion of the sewage solids takes place rapidly and energetically. This was indicated by the following facts:

1.—The sludge was well digested and drained quickly on the drying bed.

2.—No change in physical constants resulted, except a high and clear dewatering.

3.—Chemical constants indicated a better stabilization and more complete digestion.

4.—Bacterial population showed an increase for both aerobic and anaerobic.

5.—Chemical analysis of dry sludge and ash showed small variations from normal sludge.

6.—The pH-value was normal.

7.—A high rate of gas, which was mainly methane (about 70%) and CO_2 (about 30%) resulted. The calorific value was 700 B.t.u.

The entire discharge of the Rochester Gas and Electric Corporation's ammonia-still waste into the sewer leading to the (main) Irondequoit Disposal Plant† is at a lower concentration than that of the experiment. The plan has been put into operation, no upsetting of the plant has occurred, and all complaints of taste in the water have ceased.

Samuel A. Greeley,‡ M. Am Soc. C. E., and William D. Hatfield,§ Assoc. M. Am. Soc. C. E. (by letter).—The pre-aeration unit of the treatment plant at Decatur, Ill., went into operation on December 30, 1927, and a discussion of the records during the first months of 1928 is a fitting closure to the paper. The weighted operating data in Tables 19, 20, and 21 and in Fig. 14 show the efficiencies obtained by each step in the process, and the chemical content of the effluent. In Table 19, the population equivalent is the product of the 5-day bio-chemical oxygen demand and the sewage flow, multiplied by $\frac{8.33}{0.17}$. The factor, 0.17 lb. of bio-chemical oxygen demand per capita per 24 hours, has not been confirmed by independent data at Decatur, but has been accepted from the

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* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 503, Fig. 4.

† *Loc. cit.*, Vol. 91 (December, 1927), p. 524.

‡ (Pearse, Greeley & Hansen), Chicago, Ill.

§ Supt., Sewage Disposal Plant, Decatur, Ill.

TABLE 19.—OPERATION EFFICIENCIES BASED ON BIO-CHEMICAL OXYGEN DEMAND.

Month.	CRUDE SEWAGE.		POPULATION EQUIVALENT.		TOTAL PERCENTAGE REDUCTION IN BIO-CHEMICAL OXYGEN DEMAND.			
	In million gallons daily.	Five-day bio-chemical oxygen demand, in parts per million.	Crude sewage.	Final mixed effluent to river.*	By Imhoff tanks.	By pre-aeration.	By sprinkling filters.	By secondary tanks.
January	12.00	510	309 000	186 000	16.0	40.0	....	....
February	12.75	464	289 000	222 000	15.2	23.2	....	....
March	10.08	537	268 000	145 000	22.0	39.5	45.4	....
April	11.40	403	251 000	165 000	13.5	32.7	34.3	....
May	10.10	269	133 000	36 500	26.5	56.4	82.5	....
June	9.12	281	125 000	29 500	22.6	53.8	86.5	....
July	10.30	210	106 000	9 600	34.0	54.7	90.9	....
August	9.88	218	106 000	9 700	31.3	54.1	89.5	90.8
September	11.37	172	96 000	7 800	20.8	43.2	90.1	91.9
October	9.76	214	102 000	7 600	21.2	45.2	92.0	92.5
November	9.68	338	160 000	22 700	16.0	30.5	85.7	....
December	9.68	276	131 000	18 000	24.2	34.7	73.0 91.0	....

* This includes all sewage overflowed and by-passed.

TABLE 20.—OPERATION EFFICIENCIES BASED ON SUSPENDED MATTER.

Month.	Suspended matter, in parts per million.	PERCENTAGE REMOVAL OF SUSPENDED MATTER.			
		Imhoff tanks.	Pre-aeration.	Sprinkling filters.	Secondary tanks.
January	256	46.0	0	....	....
February	251	50.0	50.3	....	....
March	253	62.0	24.5	....	....
April	172	46.0	40.0	....	....
May	170	55.2	37.0	48.0	....
June	270	68.8	61.0	66.6	....
July	183	55.1	45.1	70.5	....
August	180	60.0	50.0	62.8	72.3
September	177	52.2	43.8	58.5	66.9
October	165	52.2	43.2	66.4	66.4
November	183	48.0	41.0	60.0	....
December	170	47.4	36.5	59.2	....

TABLE 21.—ANALYSES OF THE FINAL EFFLUENT FROM PLANT.

Month.	Bio-chemical oxygen demand, in parts per month.	Suspended matter, in parts per month.	Nitrate nitrogen as nitrates and nitrites, in parts per million.	Dissolved oxygen, in parts per million.	Stability to methylene blue, percentage.
January	190	162	0.4	0.0	0
February	119	103	0.9	6.0–7.0	15–60
March	84	52	3.3	0.0	37.
April	110	130	0.5	3.0	11.
May	41	89	8.1	2.8	76.5
June	38	90	11.0	3.0	84.0
July	19	54	11.0	3.5	98.
August	20	50	12.5	3.1	98.
September	14	59	9.5	4.1	98.
October	16	55	6.0	3.8	94.
November	48	73	4.7	1.5	56.
December	38	69	5.1	2.9	50.

publications of the U. S. Public Health Service and others. On a few occasions during shut-downs at the starch works the indications are that this figure is reasonably correct and applicable to Decatur. The percentage removals in Tables 19, 20, and 21, represent the total up to, and including, the treatment indicated in the column under consideration, that is, the indicated efficiency under sprinkling filters includes the removal of the Imhoff tanks plus the pre-aeration tanks plus the sprinkling filters. The percentage removal in each case is computed starting with the total population equivalent received at the treatment plant and not that actually carried through the processes. Thus, sewage by-passed during the first months of the record lowered the resultant efficiency accordingly.

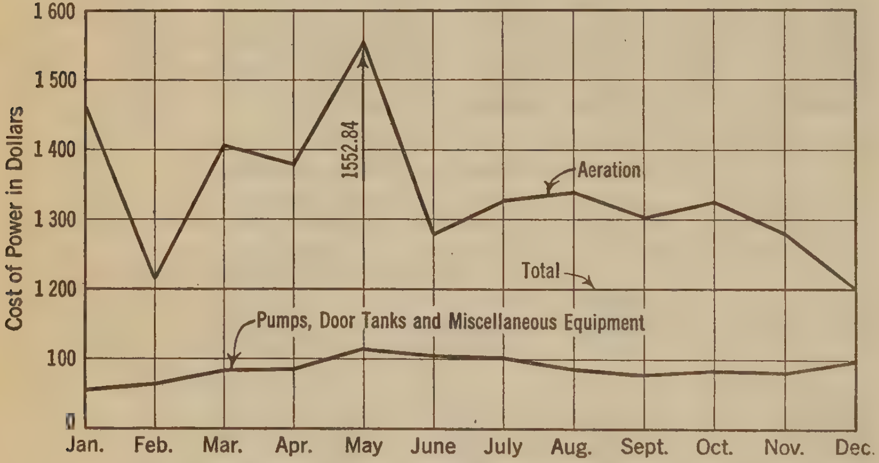

FIG. 14.—COST OF POWER FOR THE YEAR 1928.

The population equivalents of the sewage received at the plant from January through April show a gradual decrease in starch plant waste, due to more careful operation of the old process of manufacture. On April 28, the new waste recovery system went into operation. Up to this time, the pre-aeration plant (150 000 population equivalent capacity) was overloaded and was treating only a part of the sewage. This accounts for the lower efficiencies of the pre-aeration and sprinkling filter units during the early months of operation. Beginning with May, the population equivalent dropped to about 120 000, or less, and the efficiencies thereafter were satisfactory as shown by the data.

In November and December a decreased pre-aeration efficiency may be attributed to numerous factors, including lower temperatures, greater losses from the starch works, and especially the fact that one-third of the pre-aeration plant was used for sludge re-aeration. The re-aeration of return sludge varied from 9 to 18 hours and is still being studied.

The reduced sprinkling filter efficiency in November was due to the heavier loading because of stronger sewage and lower pre-aeration efficiency, while that in December was due to the fact that the filters were operated only 20 days because of high river stages. The actual efficiency of the filters during the 20 days of operation was 91%, while the over-all weighted efficiency for the 31 days was 73 per cent.

Summary.—Experience to date leads the writers to expect the following efficiencies at the Decatur treatment plant as now operated: (*a*) The Imhoff tanks remove from 20 to 25% of the bio-chemical oxygen demand; (*b*) pre-aeration raises this percentage removal to from 45 to 50; and (*c*) the sprinkling filters at 3 300 000 gal. per acre per day and secondary sedimentation further raises this removal to from 85 to 95 per cent.

The final effluent will average around 60 parts per million of suspended matter, 6.0 to 12.0 parts per million of nitrates, 3.0 parts per million of dissolved oxygen, and a stability usually greater than 90 per cent.

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Paper No. 1739

WATER SUPPLY PROBLEMS OF A DESERT REGION*

By William E. Rudolph,† M. Am. Soc. C. E.

With Discussion by Messrs. Lewis R. East and William E. Rudolph.

Synopsis

This paper treats of some of the water supply problems of a section of the Atacama Desert of Northern Chile—the sources of water, the nature of the waters and the losses to which they are subject, and the works which have been installed for impounding and transporting water. Some notes are added on the utilization of surplus water resources for irrigation and for hydro-electric power, presenting conditions peculiar to arid regions.

General

The water supply problems of desert regions are peculiar. Not only is precipitation scant in these arid lands, but the losses of water from evaporation and seepage, as well as contamination by absorption of salts, are factors which seriously affect the available supply.

Desert regions comprise nearly one-quarter of the land surface of the earth. Some have developed considerable economic importance through the discovery of mineral deposits which the very aridity of the climate has preserved. Among the latter is the Atacama Desert of Northern Chile, where the world-famous nitrate fields are located, and also important bodies of copper ore. Here, a number of inland industrial centers have grown up, as well as several prosperous seaports, among the latter being Antofagasta and Iquique, cities of more than 50 000 population.

Sources of Water of the Atacama Desert

Situated between two rain barriers, the Humboldt current and the Western Cordillera of the Andes, Northern Chile has little rainfall. At elevations below 10 000 ft., it averaged only 0.04 in. per year prior to 1925.* It is little wonder that the Desert of Atacama is one of the most barren of the earth's surfaces.

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* Published in October, 1928, *Proceedings*.

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Above an elevation of 10 000 ft. there are light showers during the summer (January to April) of most years. A single stream reaches the Pacific throughout the year—the Rio Loa (Fig. 1)—which has a maximum flow of 130 cu. ft. per sec., decreasing to 67 cu. ft. per sec., as it crosses the desert (Table 1). Yet this little stream supplies the Antofagasta nitrate pampa, the copper plant at Chuquicamata, and the City of Antofagasta, and contributes to the operation of three railroads and to the irrigation of farm lands.

In this latitude the Andes form two distinct ranges, the Cordillera Occidental and the Cordillera Oriental. Separating them is a region of high plateaus without surface drainage. The strip of land sloping to the Pacific, as well as portions of the high plateaus, form part of Chile.

The Cordillera Occidental (Fig. 1) consists of a uniformly high surface of folded and uplifted rocks, upon which volcanic masses have formed peaks and ranges. After the main range of the Cordillera Occidental had been raised to its present height, the forces of volcanism appear to have shifted toward the west, where subsidiary and lower chains of volcanic cones were built up. In but few places did they cut off drainage to the Pacific, but they did cause the formation of large lakes which gradually filled up with sediments, in some places to depths of perhaps 2 000 ft.

The waters which feed the Loa System originate primarily from summer snows and rains which fall in the high Cordillera region. These form a number of small streams (Fig. 2), most of which flow only during the heat of the day. The streams disappear within the sediment-filled lakes. Along the western bounds of these basins, at the lower points of the secondary ranges, the waters emerge as springs (Fig. 3), below which they have cut deep spectacular canyons through the volcanic formations to the lands beyond. By acting as natural reservoirs, the basins regulate the tributaries of the Loa and thus maintain a fairly constant flow throughout the day, season, and year.

East of the main range of the Cordillera Occidental are a number of wet basins without apparent outlet. Although subject to heavy evaporation, the lakes never dry up, for their water-sheds, being reached by moist east winds, experience no prolonged droughts. As they lie well above the Pacific draining region on the western side of the intervening range, there must be drainage from one to the other, probably along contacts of the older rocks and the newer volcanic topping which form this range. Thence, the subterranean waters flow directly into the basins of sediments, from which the streams of the western slopes draw most of their water. Thus, it would appear that the streams draining to the Pacific are fed from waters of the moister eastern slopes even when their local supply fails.

STREAM FLOW MEASUREMENTS

Recording stream flow in the Atacama Desert, particularly at the headwaters of the feeders, was a work fraught with difficulty. Changes in river

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* During the years 1925, 1926, and 1927, the rainfall has been greater, due perhaps to the supplanting of the cold northbound Humboldt current by the warmer southbound El Nino current during the summer of 1925. Such change in climate is probably of a transient character.

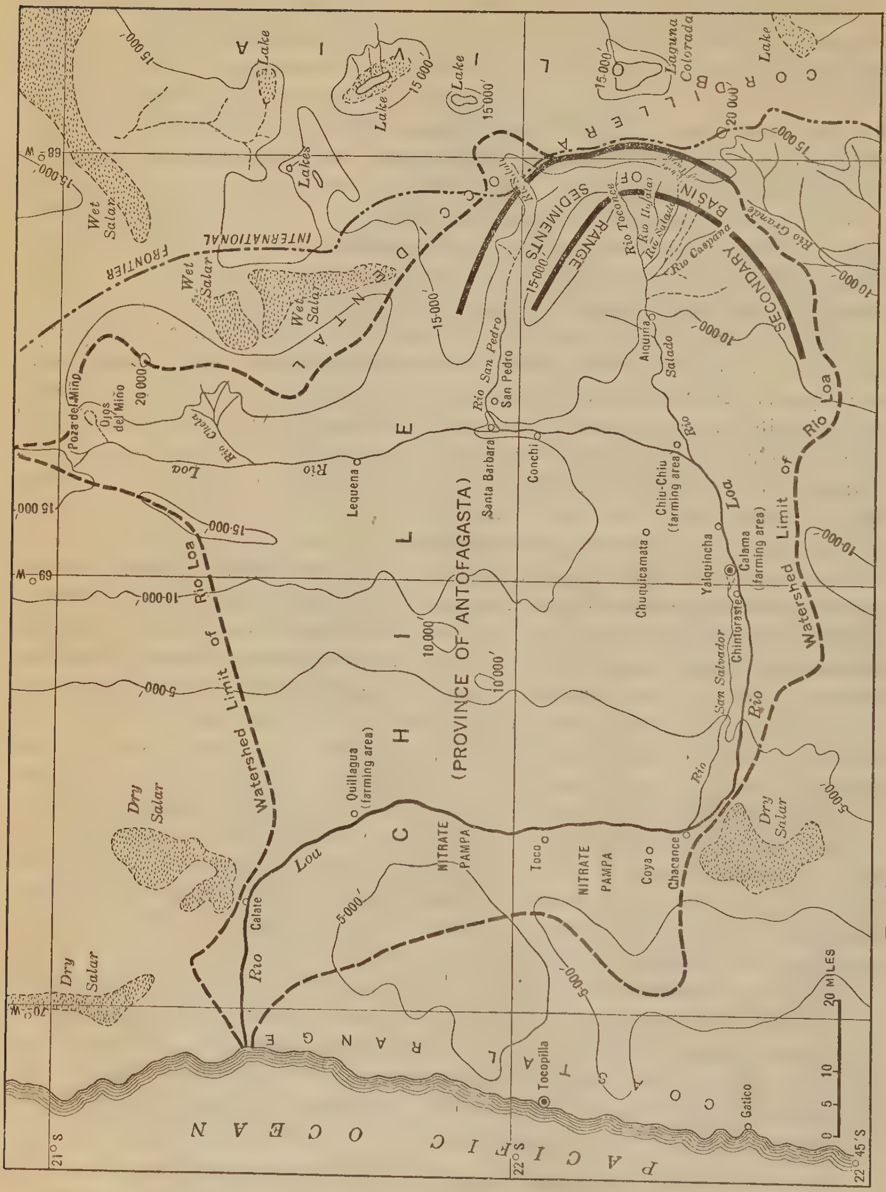
PACIFIC OCEAN
COAST RANGE
CHILE
(PROVINCE OF ANTOFAGASTA)
CORDILLERA
BOLIVIA
INTERNATIONAL FRONTIER
SEDIMENTS OF BASIN
SECONDARY RANGE
Watershed Limit of Rio Loa
Rio Loa
Rio Salado
Rio San Salvador
Rio San Pedro
Rio Toconce
Rio Hojalar
Rio Salado
Rio Caspana
Rio Grande
Rio Chela
Poza del Miño
Ojos del Miño
Lequena
Santa Barbara
San Pedro
Conchi
Aiquina
Chiu-Chiu (farming area)
Chuquicamata
Yalquincha
Calama (farming area)
Chintoraste
Quillagua (farming area)
Calate
Toco
Coya
Chacance
Tocopilla
Gatico
NITRATE PAMPA
NITRATE PAMPA
Wet Salar
Wet Salar
Wet Salar
Dry Salar
Dry Salar
Dry Salar
Lake
Lake
Lake
Lakes
Lake
Laguna Colorada
5000'
10 000'
15 000'
20 000'
68° W
69° W
70° W
21° S
22° S
22° 45' S
0 5 10 20 MILES

FIG. 1.—MAP OF CHILE BETWEEN LATITUDES 21° 0′ AND 22° 45′ SOUTH.

beds caused by flood flows permitted leakage at the sides and below weirs. Several times complete destruction followed cloudbursts. At current-meter stations the configuration of the river bed was so often subjected to alteration by high water that constant checking was necessary during the rainy season.

Observation stations were of necessity located in isolated regions, a day's muleback journey apart in many cases. Food and shelter were poor, the only inhabitants being in a few Indian pueblos. These Indians were always friendly, although the diversion of water would have been disastrous to them. The cold of the Cordillera is intense at night, whatever the season, and "soroche", or mountain sickness, was a further severe handicap to men who were not accustomed to high altitudes. The native observers placed at stations had their limitations, and supervision was both difficult and expensive.

TABLE 1.—Hydrological Data for the Rio Loa and Its Tributaries.

Streams.	Distance from source of Rio Loa, in miles.	Elevation, in feet.	Average flow, in cubic feet per second.	Minimum normal flow, in cubic feet per second.	Chlorine, parts per million.	Sulfate radical, parts per million.
Rio Loa, Upper Section:						
Rio Loa at Pozo del Miño	0	12 800	0.8	.....	Less than 100	...
“ “ Ojos “	2	12 713	8.1	.....	“ “ 100	...
“ “ Rio Chela	19	11 860	Above 11.7 Below 17.0	....	About 100	... ...
“ “ Lequena	37	10 843	21.5	19.8	146	178
“ “ Santa Barbara	57	9 810	45.9	35.3	....	...
Rio San Pedro:						
Rio Siloli, at sources	...	14 163	12.4	.....	35	10
“ San Pedro, at Santa Barbara	...	9 810	19.2	14.1	195	120
Rio Loa, Middle Section:						
Rio Loa below Rio San Pedro	57	9 810	65.1	55.0	....	...
“ at Conchi	62	9 560	70.6	61.1	....	...
“ below Chiu-Chiu Irrigation	86	8 215	60.0	49.4	403	144
Rio Salado and Feeders:						
Rio Toconce, at springs	...	13 287	14.0	13.5	62	50
“ Hojolar	...	13 878	10.6	10.0	More than 1 500	...
“ Salado, at geysers	...	14 167	14.8	14.0	4 201	48
“ Caspana	...	12 224	4.2	4.0	About 100	...
“ Salado, at Aiquina	...	9 580	45.9	37.1	More than 1 500	...
“ “ “ Chiu-Chiu	...	8 215	70.6	51.2	1 548	76
Rio Loa, Lower Section:						
Rio Loa, below Rio Salado	86	8 215	130.6	109.5	1 126	115
“ at Yalquincha } Calama irrigated area	104	7 871	123.6	102.4	....	...
“ “ Chintoraste }	111	7 005	59.0	30.0	1 292	131
“ “ Chacance	151	4 101	76.6	45.9	....	...
Rio San Salvador, at Chacance	...	4 101	26.0	22.2	1 996	264
Rio Loa below Rio San Salvador	151	4 101	102.6	68.1	....	...
“ at Quillagua (above irrigation)	206	2 733	84.8	45.9	2 088	288
Rio Loa, at Quillagua (below irrigation)	210	2 500	70.6	30.0	....	...
Rio Loa, at Calate	231	1 877	81.9	51.0	....	...
“ “ Pacific Ocean	252	0	67.0	36.0	....	...

Variations in Flow

Flow in the streams fed from underground reservoirs varies but little. For example, during 1913 monthly averages of flow in the Rio Loa just below the confluence with the Rio San Pedro remained between 56 and 62 cu. ft. per sec. Over a period of 3½ years, the limits of monthly averages were 54 and 71 cu. ft. per sec. The increases during the summer months from rains are not great. Sudden floods are occasioned by cloudbursts and heavy storms, but high water seldom lasts for more than 3 or 4 hours. In February, 1925, the volume of the Loa below the confluence with the Salado, near Chiu-Chiu, rose to more than 15 000 cu. ft. per sec. for a few hours, this being 115 times the normal flow and probably the highest in the recent history of the river.

During the irrigating season, from September to March, losses of water in the Loa below Chiu-Chiu are heavy, as may be seen from the minimum normal flows in Table 1.

LOSSES OF WATER

The flow and usability of waters of the region suffer from (1) losses from evaporation; (2) losses from seepage; and (3) absorption of salts. These three important factors will be discussed separately.

Evaporation.—Waters which enter interior drainage basins, mere evaporating pans, are lost in evaporation. Flows of the Loa and other river systems are also subject to high evaporation whenever they spread out to form marshes. Where water-works have been installed, the construction of narrow masonry canals to drain these swamp lands, or to carry the waters past them, has been necessary.

The magnitude of these losses is due in part to the extreme dryness of this region. Comparison of the evaporation observed in the Atacama Desert with that given by Ivan E. Houk, M. Am. Soc. C. E., for U. S. Reclamation projects (Table 2) show that rates in the Chilean desert are generally higher than at Yuma Citrus, Ariz., the dryest of Mr. Houk's stations. While the writer's data were compiled partly by the Chilean Government and partly by industrial companies, and, therefore, probably are not of the same high degree of accuracy as those of Mr. Houk, they are nevertheless significant.* The stations at Chacance and Chuquicamata are representative of the arid sections of the Atacama Desert.

Eventually, the water requirements of the Northern Chilean desert may necessitate complete drainage of the lakes on the Bolivian side of the frontier, by means of tunnels, to avoid evaporation losses. The costs of such works are not so apt to be a drawback as the international complications.

Seepage.—So numerous are the lava flows which have obstructed drainages and diverted them from one course to another that a detailed study of such formations may be necessary in order to determine the origin of a particular flow. Streams disappear below the surface and, later, re-appear, sometimes greater in volume, sometimes less. That underground flows are often large in quantity has been demonstrated by the increases observed in stream flow at sections where no run-off is received. Isolated water holes, ofttimes in the most arid parts of the desert, furnish further proof of such underground flows. On the face of the Coastal Range escarpment beside the Pacific, where the strata have been exposed by fracture, springs are frequent—although this is a region where showers occur perhaps once in a year. Part of the flow

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* None of the Chilean observations was based on the use of floating pans. Inasmuch as the high evaporation rates of the Atacama Desert are due to high average wind velocity rather than to high average temperature, it would seem that there would be far less difference between land and water-pan measurements in this case than at the stations mentioned by Mr. Houk.

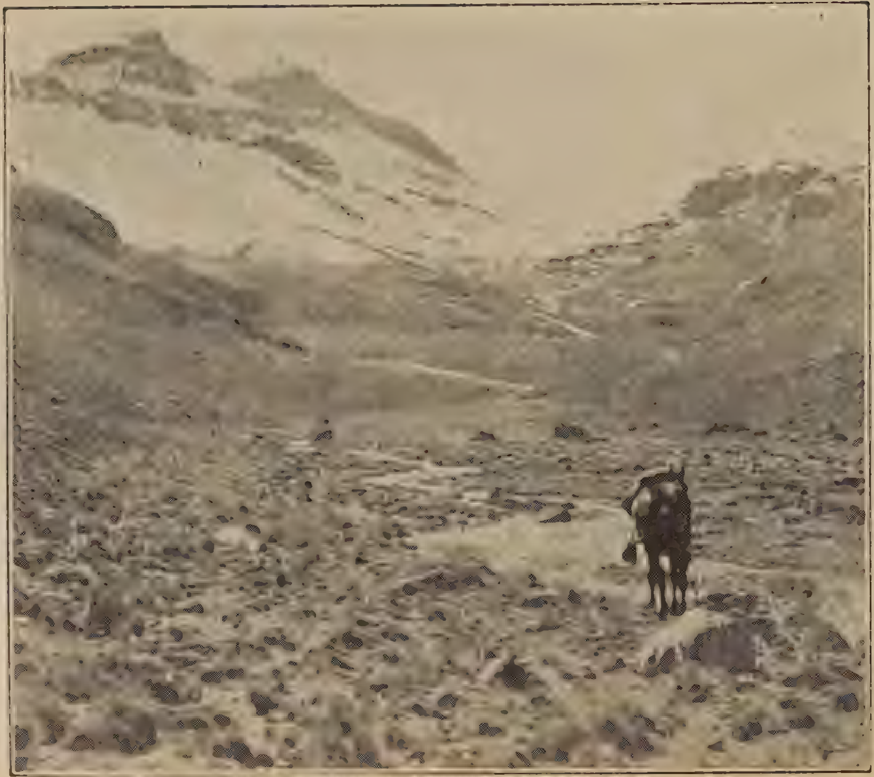

FIG. 2.—VIEW OF SMALL STREAMS ORIGINATING FROM SUMMER SNOWS AND RAINS IN THE ANDES IN CHILE.

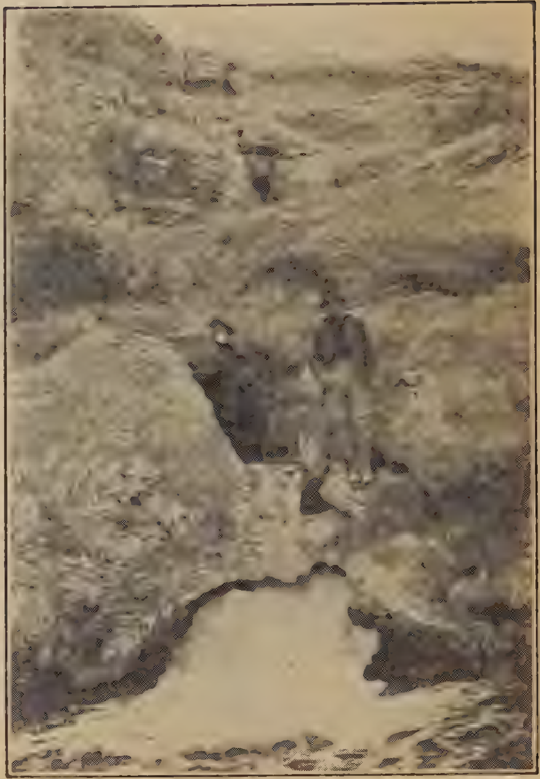

FIG. 3 —UNDERGROUND FLOW EMERGING AS SPRING IN THE ANDES IN CHILE.

of these springs is doubtless due to condensations from the afternoon clouds which envelop the coastal cliffs, but more, perhaps, comes from underground passages extending a hundred miles or more inland to the Cordillera region.

TABLE 2.—COMPARISON OF EVAPORATION RECORDS OF UNITED STATES RECLAMATION PROJECTS* AND OF ATACAMA DESERT, CHILE.

Station.	Years.	Annual evaporation, in inches.	Mean temperature, in degrees Fahrenheit.	Mean wind velocity, in miles per hour.	Remarks.
Yuma Citrus, Ariz.	1921-23	122.81	70.7	3.03	Station 8 miles southwest of Yuma on desert mesa.
Yuma Citrus, Ariz.	1921	135.70	70.7	3.03	Maximum during 3 years, 1921 to 1923.
Elephant Butte, N. Mex.	1917	109.70	61.1	4.52	Maximum during 7 years, 1917 to 1923.
San Pedro	1917 and 1918	117.15	About 50	About 10	In mountain region subject to annual precipitation of about 1.7 in.
Chuquicamata	Various observations, 1917-27	144.00	" 58	9.81	In arid desert, very little precipitation.
Calama..................	1917	113.40	" 57	Ranging between 6 and 8	Within irrigated area, very little precipitation.
Chacance	1917 and 1918	181.00	" 65		In arid desert, rainfall minimum, about once in 2 years.
Quillagua	1917	119.00	" 65		Within irrigated area, rainfall minimum, about once in 2 years.

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* From paper by Ivan E. Houk, M. Am. Soc. C. E., entitled, "Evaporation on United States Reclamation Projects," *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 266.

The underlying rock formations are of such structure as to cause loss of much of the water passing through them. Some of the waters that wander away through limestone passages or surface sediments, are certain to return to the main channel because of the water-tight flooring of hard crystalline rocks. On the other hand where such waters have access to underlying stratified rocks of a permeable nature, heavy losses result, due to the steep slopes which facilitate subterranean flow direct to the ocean. It is difficult to trace waters lost in the pervious strata of sandstone because the overlying mantles of detritus are sometimes hundreds of feet in thickness.

Thus far, losses due to seepage have not called for considerable attention because of the moderate water requirements. With increased exploitation of the mineral resources of this region the engineer must face the task of reclaiming these waters or of preventing their escape. As far as the writer knows, no deep borings have been undertaken to reach the underground flows, which probably become concentrated within buried drainage systems of the past.

Absorption of Salts.—Some of the volcanic barriers through which the flows pass contain considerable salts of uncertain origin. Perhaps they were caused by decomposition of volcanic tuff due to fumarolic waters, such as still exist in the region. Again, they may be from vast beds of sodium chloride which crystallized out in basins as the continent was elevated more than two miles out of the sea, later to be covered with volcanic materials. Whatever the cause, many of the waters that are chemically quite pure near their sources, acquire so much salt while passing through volcanic formations, particularly at the lower boundaries of the basins of sediments, that they cease to be potable.

Chemical concentrations are highest in waters of geysers and boiling springs. The geysers of the Tatio initiate a flow of 14.8 cu. ft. per sec., with a chlorine content of 4 200 parts per million. The Chilean Government once considered diverting this river to an evaporating basin outside the Rio Loa System, but a study of the economics of such an undertaking caused it to be abandoned.

Seldom are waters of the desert so chemically pure as those ordinarily used in regions of normal rainfall. Only at the upper reaches of the Loa System (Table 1) is the water low in chlorides and sulfates. In the United States, waters containing chlorine in excess of 20 parts per million are not desirable because of corrosion to boilers and plumbing fixtures. In Northern Chile chlorine tolerance for potable waters and those used for generating steam is usually set at 50 parts per million, and the Chilean Government has stretched the point slightly by including within this category waters of the Rio Loa System which have as much as 62 parts per million.

Whereas waters used in steam boilers may be treated for sulfates and carbonates, the writer knows of no practicable way of eliminating chlorine. The waters of the Rio Toconce, containing 62 parts per million of chlorine, have been used for a number of years in locomotives. The Antofagasta-Bolivia Railroad Company at one time used the water of the Rio San Pedro having 195 parts per million of chlorine, but had to obtain other supplies. For drinking, waters similar to those of the Rio San Pedro are apt to have a mild purgative effect. Those having as much as 1 000 parts per million of chlorine are not fit to drink. The people of the Village of Quillagua, although very poor, purchase drinking water from the Railroad Company at the equivalent of 1¼ cents (U. S. currency) per gal., rather than use the brackish water of the Loa.

For irrigation the Chilean Government classifies as good, waters not exceeding 200 parts per million in chlorine. In general, only the Indian communities near the head-waters of the region have such water. Nevertheless, good crops of alfalfa, corn, and some vegetables are raised with water of from two to ten times this maximum.

Streams which spread out to form marshes are naturally high in salinity. The salt content of most of the waters of the region is also increased 50% or more during the summer season, when the rains in the upper sections dissolve saline efflorescences.

In the basins without outlet, the lakes have a widely diversified chemical content, including alkalis and borax. The impotability of many of these waters is attested by the bones of animals along the shores. In other lakes, as the Laguna Colorada, the brackish water forms a large pool in the center, separated by crystallizations of salts from the outside waters, which are potable.

Intakes for Potable Water Supply

In view of what has been said about evaporation and seepage losses as well as the acquisition of soluble salts, apparently potable water intakes might best be located at the upper sources of the feeders, that is, at the snow drainages, before they disappear into the reservoirs of sediment. This is seldom feasible, however, because of (*a*) the high altitude and general inaccessibility of such places; (*b*) the intermittent character of flow due to freezing; and (*c*) the fact that in very dry years there may be no flow at all.

In a few cases snow drainage water, although flowing underground, has been piped by building subterranean dams—such works being limited to narrow valleys. At others, the waters have been taken from rivers at elevations between 11 000 and 14 000 ft. Tunnels driven into sandstone formations to collect and divert ground-waters to intakes have also been resorted to, but they seldom yield as much as natural springs. For example, the fifteen canals of the Pica Region, Province of Tarapaca, discharge but little more than 1 cu. ft. per sec., despite their 8 miles of length.

An advantageous location for the intake is at the springs where the waters issue from the natural reservoirs. They may have already dissolved some salts, but not in large quantity. Small canals carry the water to the settling basin at the intake of the pipe line (Fig. 4).

Where waters are removed from streams, the diversion dam and intake chamber require merely sufficient height to keep the pipe line from drawing air. Constant cleaning is necessary, for not only is there considerable sediment, but also floating matter, chiefly pebbles of pumice, washed from the ground by the rains. Pollution by sewage seldom calls for consideration in locating a potable water intake, for the water sources are rarely inhabited.

Pipe Lines

Pipe lines depend on gravity in the upper sections and pumping stations in the lower. The pipes are nearly always of steel or cast iron; wood-stave lines, although cheaper, are hardly safe in regions of the desert where water is scarce and scruples are few. One structure was made of hard durable metal on purpose "to resist destruction by travelers who ordinarily take such objects as targets for revolver or rifle shots."

These lines are laid on the ground with a covering of about 15 in. of soil, except at the joints. This protects them against temperature changes which sometimes amount to 80° Fahr. within 12 hours in Northern Chile. The pipe line full of water is not as a rule affected by temperature, but when a break occurs difficulties may ensue. Pipe lines are covered as they are laid, using expansion joints to take up such temperature stresses until the earth covering becomes compact.

In many cases the water is conveyed for long distances, the supply of the City of Antofagasta traveling 230 miles. Some of the lines are operated under but little pressure, open tanks being located at intervals. Others, which cross low valleys, necessitate pressures as high as 1 000 lb. per sq. in.

The design of pipe lines ofttimes presents complications. Transportation of materials must be considered in the location (Fig. 5), for, in country covered with lava flows and cut by deep canyons, delivery costs may far exceed the value of the pipe.* Also, important is the hydraulic grade, particularly in the section just below the intake where the line must generally be located within the bounds of a deep precipitous canyon. Here, construction expenses are high. Yet to reduce them by resorting to a minimum grade for the pipe line is apt to be short-sighted, due to the tendency of the hydraulic grade to fall below the level of the pipe after several years of operation, with diminished capacity and other troubles resulting. The writer recalls one instance wherein it was necessary to lay "looping" lines after eight years of operation, in order to bring a pipe line to the minimum capacity for which it was designed.

OPERATION

The high chemical content of the water is apt to make pipe corrosion problems more acute in desert regions than under similar conditions in humid lands, for sulfates and chlorides, usually so abundant, aid the attack on iron. In Northern Chile, analysis of nodular incrustations found in a pipe line showed a composition of 0.8% sulfate radical and 0.2% chlorine, a concentration which is quite remarkable considering that only 50 and 62 parts per million of these radicals occurred in the water itself. Ground-waters of regions of recent volcanic activity, where rainfall is scant, are apt to contain large quantities of carbon dioxide and hydrogen sulfide, both gases being agents of corrosion. The warmth of spring waters (often as high as 75° Fahr. throughout the year) also contributes to their corrosive properties.

Because of the corrosive quality of these waters, it is highly important that air be excluded from pipe lines. When the hydraulic grade line falls below the level of the pipe, even for transient periods as when scouring is done, water wasted, or branch lines are fed, air is admitted and conditions favorable to corrosion are set up.

For removing sand and sediments, sand traps are best, as they do not cause the impairments so often attendant upon scouring. Valves for emptying a pipe line are necessary, as for discharging the water when repairs are in progress; but when such valves are to be used for scouring, their locations as well as the manner of their operation call for careful study. One company used floating balls of hollow metal for ridding its line of sand. The ball was made about 1½ in. smaller in diameter than the pipe. When its progress was checked by accumulations of sand the pressure rose and the obstruction was removed. At the same time the ball was not so large as to endanger the line by completely blocking the flow. Its passage could be checked by the noise which it made—in fact, a dog was trained to follow and bark at it.

Automatic air-release valves, located at summits where the pressure is low, serve to eject some of the air entering the line. With cold non-corrosive water, and steady flow maintained under pressures greater than atmospheric,

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* Mules had been used to a great extent in the past for haulage and distribution. In more recent years, motor trucks, tractors, and trailers have effected considerable savings over the older methods, despite the initial expenses for road building. In soft sandy portions of the desert, caterpillar trailers have been used to good advantage.

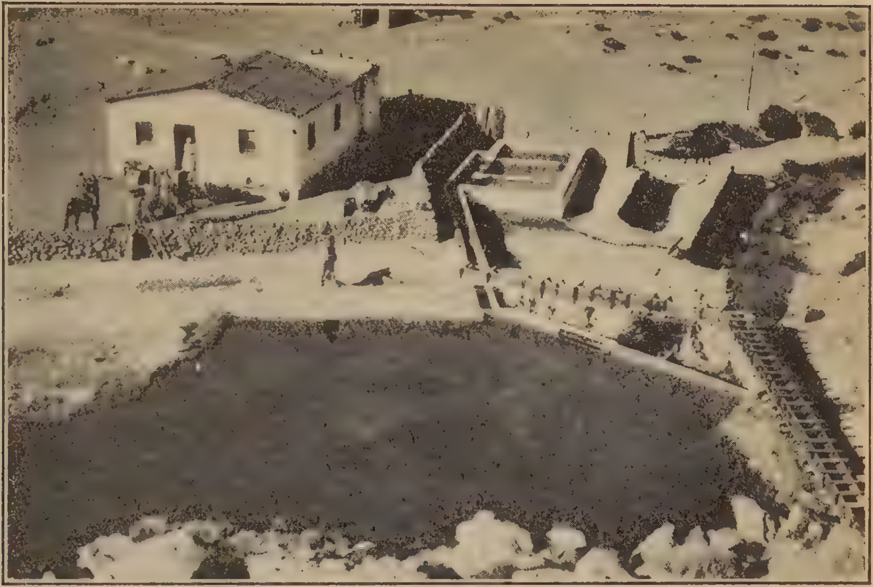

FIG. 4.—VIEW SHOWING SMALL CANAL CARRYING WATER TO SETTLING BASIN AT INTAKE OF PIPE LINE.

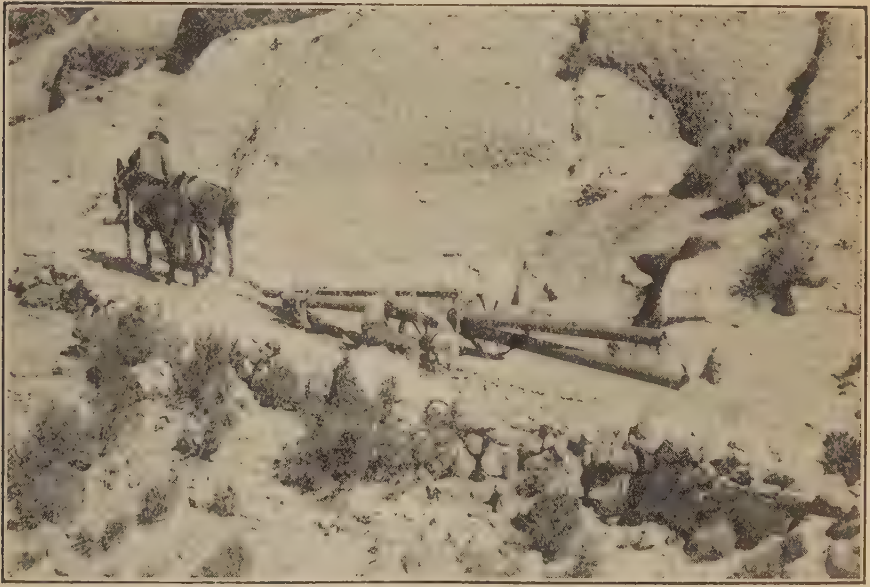

FIG. 5.—METHOD OF TRANSPORTING MATERIALS FOR PIPE LINES IN CHILE.

deterioration of a long-distance water main will be slow. With corrosive water, it may be well to install works for de-aerating the water or rendering it non-corrosive at the source.

Some protective coatings appear sufficiently substantial to safeguard a line against corrosion. However, until a coating is found combining low cost with durability it will probably be more economical to take measures to exclude free oxygen from pipe lines.

The type of joint is another matter which is important. Too frequently the common screw coupling provides most favorable conditions for storing air, with consequent chemical injury.

Other Means of Transportation of Water

Many of the villages are supplied from tank cars, the inhabitants making daily trips to the station to fill their cans. Another means of distribution is the mule cart, with its daily rounds of nitrate district villages as well as smaller pueblos. In more isolated districts, one sees mules and burros carrying potable water, often for 20 or 30 miles. Whereas 26 gal. per day per inhabitant within cities, and half that quantity outside of cities, are the quotas on which the Chilean Government bases concessions for drinking water supplies in its desert provinces, the actual use of water is generally less. This is due to the extreme poverty of the people. Large industrial companies have inaugurated the practice of furnishing free water to workmen and their families, as well as providing sanitary toilets and bath houses for their use.

Irrigation

A potent argument for conservation of water resources in desert regions under development lies in the needs of agriculture. Where food is scarce and expensive, with increments of population continually being brought in to work at mines and industries, all surplus water is required for irrigation. This applies not only to flows lost through seepage and useless evaporation, but also to those that reach the sea.

In Northern Chile large water resources are still being lost or wasted. Indian dwellers of the highlands have installed irrigation canals, marvels of ingenuity, within the canyons of the upper drainages, but these produce only sufficient for their owners' needs. Farming centers in sections of lower altitude cultivate alfalfa in quantities, but only small quantities of foodstuffs. Meanwhile, imported food fills the shelves of every store, and that from Central and Southern Chile has to be transported about 1 000 miles.

That irrigation can produce successful crops has been demonstrated at Chiu-Chiu, Calama, and Quillagua, within the Rio Loa Basin. Yet this river pours 36 cu. ft. per sec. into the sea during the farming season. Once there was a project—work had actually been begun, in fact—to enlarge the cultivated area at Quillagua. The plan came to naught when the earthquake of 1879 destroyed some of the completed canals. It can be only a question of time before foreign capital interested in the nitrates and minerals of this district will take the matter in hand.

HYDRO-ELECTRIC POWER

In a region where water is scarce it would be thought that hydro-electric power possibilities would be of little consequence. This does not necessarily follow, for a meager supply may be balanced to some extent by the high head available.

With present low prices for fuel oil, developments of the hydro-electric resources of the Loa and adjacent basins do not appear attractive. Yet such potential installations, costing between $450 and $500 per h.p., require no considerable rise in fuel costs before they assume importance. A few water-power plants, generating about 500 h.p. per installation, have been in operation in the lower sections of the Rio Loa for some years, in connection with the nitrate industry.

As a source of energy, however, the harnessing of the Loa and adjacent waters has hardly been begun. Projects at the upper feeders of these rivers are unique in that they call for series of power houses using the same water again and again. The Rio Salado and its tributaries may be utilized through a head of 5 000 ft. in 35 miles above its confluence with the Loa, developing about 8 000 h.p. Discharges from springs of as little as 0.1 cu. ft. per sec., when at the higher elevations, represent important potential energy and in many cases are well worth diverting to the canals.

Storage reservoirs are seldom advantageous, due to the constancy of stream flows and to the high rate of evaporation. The diversion dam at the intake of the canal requires only sufficient height either to meet the requirements of diurnal variations, due to freezing of tributaries in the upper regions, or to regulate flow below sections when irrigation is carried on during the daylight hours, in the lower regions.

A contributing factor in making these projects so expensive is the porous nature of many of the rock formations. Lined canals are necessary for the most part, particularly in view of the scarcity of water. The volcanic rock formations of the mountain region are peculiar in that the hardest and most durable often occur at the surface, so that erosion has formed overhanging cliffs at canyons, with massive fallen fragments among the talus beneath. Needless to say, such conditions call for expensive construction to safeguard water-works.

Only above an elevation of 12 000 ft. need power-house roofs be designed for snow load. Frost does not affect foundations carried 2 ft. below ground.

Some water supply pipe lines have been designed to furnish energy for operating power plants in this region. A small emergency plant, installed near the outlet, is quite certain to justify its cost, provided the pipe line is not overstressed under the increased pressures.

DISTILLATION OF OCEAN WATERS

Distillation of ocean waters is another source of supply. The seaport of Tocopilla obtains its water in this way, at a cost of $0.60 (U. S. currency) per ton, and more. With a prosperous period for the Chilean nitrate industry and consequent growth of this port, a pipe line from the Cordillera region would seem necessary.

This method is out of the question for the nitrate and mining centers as these are situated at elevations of 3 000 ft., or more. A drinking supply for a large number of these establishments is obtained by distillation of brackish river waters.

CONCLUSION

It has been seen how the projecting of water-works in a desert region may involve a great variety of problems, from the utilization of snow waters at 3 miles above sea level to the distillation of waters of the ocean itself. So scant is the supply in the Atacama Desert of Northern Chile that, with expansion of industry, the engineer is apt to be delving within the realms of geology and chemistry in order to find ways of conserving the quantity and quality of the water available.

DISCUSSION

Lewis R. East,* Assoc. M. Am. Soc. C. E. (by letter).—Australia has a land area of 3 000 000 sq. miles, which is slightly greater than that of the United States. It is, however, less favored in the matter of rainfall and, in consequence, is peculiarly destitute of great rivers. Fortunately, over large areas, underground water supplies, of good quality, can be obtained at reasonable depths. In the arid and semi-arid regions, considerable work has been done in providing water supplies for domestic, stock, and irrigation purposes.

Apart from comparatively short coastal streams, the only rivers of consequence are the Murray and its tributaries, which, during the winter months, have a total navigable length of approximately 3 212 miles. The Murray Basin has an area of 414 000 sq. miles, of which only 159 000 sq. miles can be regarded as contributing to the river flow, and some of this to a small degree only. Part of the catchment has a heavy rainfall, and some of the mountainous areas are covered with snow for several months in each year. The average annual rainfall over the whole area is, however, only 13 in.

Evaporation losses appear, from observations, to range up to a maximum of about 60 in. per annum. In a year of low rainfall, the total discharge of the Murray River System has fallen as low as 2 327 000 acre-ft., while in a wet year the run-off is more than twelve times this amount, the average being 9 328 000 acre-ft.

It has long been realized that storages are essential for the economic use of available supplies, and—especially in Victoria and New South Wales—all political parties for many years have followed progressive policies in regard to water conservation.

On the Murrumbidgee River, one of the tributaries of the Murray, a cyclopean concrete dam 200 ft. high was completed in 1925 to impound 772 000 acre-ft. of water, and on the Upper Murray good progress is being made with the Hume Reservoir which, when completed, will store 2 000 000 acre-ft. This storage will rank with those of the Gouin, Elephant Butte, and La Boquilla Dams, among the world's largest artificial storages. On the Goulburn River and other tributaries, storages have been completed with capacities totaling 800 000 acre-ft., and other works are under construction which will bring the total storage on the Murray and its tributaries to more than 4 000 000 acre-ft. The waters thus conserved on the Murray and other smaller river systems in Victoria are being used largely for irrigation; but they will also be used for domestic and stock supplies on several millions of acres outside the irrigation areas.

The State of Victoria under the administration of the State Rivers and Water Supply Commission (corresponding to the Bureau of Reclamation, of the United States) already has made considerable progress in water supply and irrigation development. It should be remembered that one of the first irrigation plans in Australia—and certainly the earliest, involving high-lift pumping on a large scale—was inaugurated in 1887 by Mr. George Chaffey who later played such an important part in the development of the Imperial Valley, California.

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In this discussion, the writer wishes to describe some of the problems of domestic and stock water supply in arid and semi-arid parts of Australia, and the methods by which these problems have been solved. This will be confined to a description of the various water-works supplying the northwest part of the State of Victoria, in Southeastern Australia, over which the mean annual rainfall varies from 20 in. in the more favored Wimmera Area to 12 in. in the far north, or Mallee Area.

Geology.—The geology of the Mallee Area has an important bearing on its water supply. In geologically recent times it was covered by a great inland bay into which discharged the head-waters of what are now the rivers of the Murray System. Later, as this bay became entirely cut off from the sea by drifting sands and marsh growths, lacustrine conditions developed, and the depression was gradually filled with sands and gravels brought down from the surrounding ranges. The whole area thus became a vast sandy plain over the margins of which the streams continued to pour their burdens of silts, sands, and gravels.

The materials laid down before and after the area was raised are entirely distinctive—the latter, or fluviatile, being gray, and the former, or lacustrine, being red. While the fluviatile deposits were gradually extending over the edges of the lacustrine area, the surface of the remainder, under semi-arid conditions, was being blown broadcast by wind action until the whole of the exposed lacustrine area had been redistributed. The Mallee country now consists of low sand ridges interspersed by harder flats, generally with an east and west tendency. The larger streams have joined to become the Murray System and have cut their way through these deposits exposing cliffs varying from 60 to 180 ft. in height. The lesser streams from the south "lose themselves" in the sands and do not reach the Murray. Many salt lakes remain in low-lying sections, but throughout the Mallee (the Victorian Area is 11 000 000 acres), there are no defined watercourses other than the Murray Canyon itself.

Natural Water Supplies.—As may be expected, natural surface water supplies are exceedingly scarce and unreliable in the Mallee country, which up to comparatively recent years was shown on maps as "sandy desert covered with dense scrubs of *Eucalyptus Dumosa*". This *Eucalyptus Dumosa* is the Mallee scrub which gives its name to the area. After heavy rains, surface water can frequently be obtained where, under favorable conditions, rain water collects and lies for months in depressions or clay pans. "Soaks" can also be located by experienced observers at places where ground-water can be obtained by digging a few feet. The aborigines were adepts at this, and many native wells have been found. Mallee exploration was attended by great hardships due to lack of water. Historical records tell of one of the early surveyors who reached the river only after killing his horse and drinking its blood.

The first efforts of early settlers were naturally to develop available "soaks" and catchments. Where water collected naturally at clay pans, a hole was sunk and lined with pine logs (a native tree, *Callitris Robusta,* which is not the true pine). This was roofed over to reduce evaporation, and it held small quantities of water for domestic use over long periods of dry weather.

Shafts or Wells.—In certain localities near the mountains, shafts were sunk to underground water—sometimes to depths of several hundred feet in extreme cases—and timbered. Water was raised by means of horse works, windmills, or engines.

Catchment Tanks.—Catchments are sometimes natural, that is, open spaces with hard ground; but generally they are prepared by clearing suitable slopes and plowing complete systems of catch-drains leading to excavated tanks which must be located in good "holding ground", that is, in clay. Catchment tanks vary in depth according to the depth of clay available, and excavation must not go through the clay to underlying porous drifts. Side slopes vary from 1 on 2 to 1 on 3, and for average depths, up to 15 ft., the cost of excavation by scoop teams is about $0.30 (15d) per cu. yd.

It has been found that, with a 12-in. rainfall, approximately 20 acres of catchment are required for each 1 000 cu. yd. of storage (170 000 Imperial gal.). The actual percentage of run-off is very low, because a 0.5-in. catchment is rare, that is, a catchment which will run water with a 0.5-in. rain. Practically all supplies are obtained from occasional heavy thunder-storms of more than about 1 in. Trampling of the catchment by grazing stock is advantageous, but cultivation, of course, cannot be permitted. The minimum storage capacity for a wheat farm of about 1 sq. mile in area, is about 2 000 cu. yd. which requires about 40 acres of catchment. The best catchments are reserved for public tanks and the largest tank thus far constructed and filled in the Mallee has a capacity of 40 000 cu. yd. When Mallee lands are thrown open for selectors, an addition of from $0.73 to $0.97 (3s to 4s) per acre is made to the purchase price to pay for preliminary road and water supply works. This addition or "loading" is paid by the settlers at the same time as land installments over a period of from 20 to 40 years. The full amount of the loading is, therefore, not available immediately for works; but the present value—approximately three-fifths—is spent in constructing public catchment tanks and in grubbing roads for access to the farms.

To reduce evaporation losses which are considerable, tanks are sometimes covered by roofing with wire netting overlaid with dry grass tussocks to a depth of from 1 to 2 ft. Usually, only the edges are covered, the center being left open. This is found to be quite effective.

Rock Catchments.—Mention should be made of the "rock catchments" of Western Australia where outcrops of solid granite have been surrounded by cement masonry drains leading to adjoining excavated storages. There are in some parts of this State catchments of several hundreds of acres in area and 150 ft. above the general level of the surrounding country. These rock catchments yield as much as 95% of the first inch of rain and practically all of any rain following immediately.

Storages adjoining the catchments are generally excavated to a depth of about 15 ft., the bottom 10 to 12 ft. being water-tight clay, above which there may be from 3 to 5 ft. of porous material. Loss of water is prevented by excavating a trench to clay under the site of the spoil-banks and filling this with clay pug.

Bores.—Below the æolian surface soils of the Mallee and red lacustrines there is a more or less impervious blue clay band under which are considerable thicknesses of marine tertiaries, containing water under artesian conditions. This water is invariably mineralized to a greater or less extent—sometimes approaching saturation—and the minerals in solution are as invariably in the proportions of sea water. There are generally several layers of water at different depths, that is, perched basins, and the top water is usually of very inferior quality. In the southwestern part of the Mallee adjoining the South Australian boundary, usable water containing about 70 parts of dissolved salts per 100 000 is obtained by boring to depths of from 100 to 200 ft. This water rises to within about 50 ft. of the surface. At greater depths of from 450 to 550 ft., water containing as little as 35 parts of dissolved solids per 100 000 has been obtained, and as much as 6 000 gal. per hour has been pumped from a bore, 6 in. in diameter, at this depth without unduly lowering the level of water in the bore. To the east of this district the depth to usable water increases and the quality of the water deteriorates until in about 60 miles a limit of about 500 parts per 100 000 is reached. Farther east the quality is even worse.

Limits of salinity (total dissolved solids) vary according to the nature and proportions of the salts in solution, but in the Victorian Mallee (sea-water proportions) may be taken as follows:

Human consumption	Good	30	parts	per	100 000;	0.05	oz.	per	gal.
	Medium	60	"	"	" ;	0.10	"	"	"
	Bad	150	"	"	" ;	0.25	"	"	"
	Limit	620	"	"	" ;	1.00	"	"	"
Working horses	Good	300	"	"	" ;	0.50	"	"	"
	Fair	400	"	"	" ;	0.625	"	"	"
	Limit	500	"	"	" ;	0.80	"	"	"
Grazing stock	Limit	900	"	"	" ;	1.50	"	"	"

Under inland conditions sheep have been known to do well on water containing 2 oz. to the gallon when they drank it continuously.

The area under which usable water can be obtained covers about 1 000 000 acres of agricultural lands. In this region the State Rivers and Water Supply Commission has driven nearly 100 bores, each supplying water for eight or nine farm holdings of 700 acres. The bores (mostly 5 in. in diameter) are cased so as to cut off waters of inferior quality, and, in general, are equipped with windmills 12 ft. in diameter on 30-ft. steel towers. Storage is provided by 4 000-gal., corrugated-iron tanks on timber or steel stands. The cost of a bore complete with mill and tank averages about $1 200 (£250), the actual boring costing about $2.43 (10s) per ft.

As the water is generally neutral or slightly alkaline, the casing has a long life; no failures due to its corrosion are known in this area although many of the bores have been been operating since before 1914. The entire cost of constructing and maintaining the bores is met by rating the lands that are benefited; the actual rate averages about $0.04 (2d) per acre per annum. The cost of maintenance and repairs is about $100 (£20) per bore per year.

Channels.—Gravitation Supplies.—The bore area is limited. Catchment tanks, which completely failed in times of drought, proved inadequate to meet the demands made on them as the country developed and the urgent necessity of providing assured and ample water supplies for working and grazing stock has resulted in the development of an extraordinarily extensive system of channels bringing water from the wet south to the dry north.

As might be anticipated from its geological formation, the Wimmera-Mallee Area has a general northerly slope from the southern water-shed—the Grampians and the Dividing Range. From these ranges a number of small rivers cross the Wimmera plains and disappear in the sandy soils of the Mallee. None of them reaches the Murray or the sea, even in times of great flood. As early as 1880, the natural flow of the chief of these streams, the Wimmera River, was diverted into channels by means of timber weirs and carried to excavated tanks on farms in sections of the country where there were no defined watercourses. By 1891 about 500 miles of these channels had been constructed. The only storage at this time was Wartook Reservoir of 24 000 acre-ft. capacity. A further storage (Lake Lonsdale, 45 000 acre-ft.) was completed in 1902 at the end of a drought year in which the whole distribution system practically failed. After the constitution of the State Rivers and Water Supply Commission in 1906, all these works came under Government control and considerable extensions were made.

The old method of using creeks and watercourses as main channels resulted in excessive evaporation and seepage losses. The Commission, therefore, abandoned all natural depressions and laid out a system of main channels suitably located along main ridges to command practically the whole of the Wimmera and Mallee Districts. Water from storages has been taken across rivers by means of low-level weirs or diversion dams which not only enables it to cross the streams, but also permits any river flow available to be diverted into the main channel. In normal seasons the river flow is sufficient to keep the main channel full throughout the winter and spring without drawing on the storages which are thus kept in reserve to make up deficiencies during periods of little or no flow.

Since 1906 the area served by the Wimmera-Mallee Gravitation System, has been more than trebled until at present (1929) about 11 000 sq. miles are supplied from its channels. To serve this area, which includes 35 townships and has a population of 100 000, more than 5 100 miles of channels have been constructed by the Commission. The longest Government channel carries water a distance of 300 miles from the storages in the hills into the far north. This great extension of the original plan has necessitated full development of the catchments of all streams supplying the area and the total capacity of storages is now 200 000 acre-ft. Slightly less than one-half of this is required for watering the whole area in an average year.

The channels vary in capacity from 700 to 4 cu. ft. per sec., and their grades also vary from 10 ft. per mile to 9 in., depending on the fall of the country and the size of the channels. The velocity of flow is usually kept about 0.5 ft. per sec. Channels in cuts as deep as 4 ft. have side slopes of 1 on 1, and for greater depths, 1 on 1.5. In very sandy land, such as that in desert country, the channels are lined with about 1 ft. of clay or loam to minimize seepage losses, but in general they are merely excavated in the earth. In designing channels for domestic and stock water supplies, it is usual to allow 1 cu. ft. per sec. for every 40 sq. miles, but no main channel should be designed for less than 10 cu. ft. per sec. and the smallest distributary (bed width usually 3 ft. for reasons of construction), should be not less than 4 cu. ft. per sec. The extreme length of a small channel of this size should not exceed 10 to 15 miles. For longer channels the capacities must be increased, regardless of the actual area served.

It must be remembered that no irrigation is permitted from the channels of this system; all the water available is required to run the channels, in which seepage and evaporation losses are very great, and to fill excavated storages on each farm once per year, with enough water to serve all needs for the full twelve months until the next run of the channels. As far as possible the channels are run during the winter months in order to reduce seepage and evaporation losses. Farmers are required to provide tanks on their holdings of a total capacity equivalent approximately to 4 cu. yd. for each acre; that is, 2 500 cu. yd. for a 640-acre holding. The minimum depth required is 12 ft. and, as channels are generally on high ground, storage above ground level is frequently possible. Excavation is, therefore, often only two-thirds of the full capacity, the remainder being carried above ground level by banks. The minimum supply necessary for an average farm of about 640 acres is about 750 cu. yd., or 120 000 Imperial gal. per year. This will suffice for about 200 sheep, 6 cattle, 12 horses, and 10 persons. An open storage of 2 500 cu. yd., or 425 000 Imperial gal., will yield this quantity after allowing for all evaporation losses and waste during the year.

As in the bore areas, the landholders are rated on a valuation basis to cover the cost of the project. The average rate is from $58 to $100 (£12 to £20) per sq. mile, with a minimum of about $52 (£10/13/4) per sq. mile, or $0.08 (4d) per acre. The charge is not based on the quantity of water supplied. To encourage the landholders to put down storages of ample capacity the Commission charges no more for filling a 15 000-cu. yd. tank than one of 2 500 cu. yd. Under favorable conditions 200 miles of channel can be run in 30 days, but in more porous country the time required might be 60 days.

Maintenance consists largely in removing sand-drift blown into channels during hot windy weather. In the parts of the Mallee particularly liable to "drift", channels are occasionally entirely filled with sand blown by one storm. Thirty-five townships within the area commanded by the Wimmera-Mallee System have pipe reticulations supplied from local storages which are filled annually from its channels.

The project has cost $8 750 000 (£1 800 000) to date, or a little more than $100 per acre, and is proving a sound financial proposition to the State apart from its general contribution to the prosperity of the country.

Channels.—Pumped Supplies.—In addition to this great gravitation system the Commission has completed several independent projects by which large areas are supplied with water for domestic and stock purposes through channels filled by pumping from the River Murray. In the County of Millewa approximately 1 000 sq. miles are thus supplied through separate low-level and high-level channel systems. Three pumping plants are required to serve the lower system, which is approximately 145 ft. above river level. The main lift is 103 ft., and the minor lifts, 13 ft. and 19 ft., respectively. The high-level system requires a further lift of 135 ft., thus raising the water a total of 270 ft.

For the low-level system (known as the Lower Millewa Water-Works District), the average rate is $0.16 (8d) per acre, or $103 (£21/6/8) per sq. mile. For the high-level (or Upper Millewa District), the rate is $0.20 (10d) per acre, or $129 (£26/13/4) per sq. mile. The length of public channels constructed in the Millewa Districts to date is approximately 700 miles.

Pipe Reticulation.—In the State of Victoria the supply of water to farms through pipe reticulations has not been extensively adopted owing to its great cost. However, in the adjoining State of South Australia, several such layouts have been in operation for a number of years and one, the Tod River Water Scheme,* when completed, will have in its 240-mile main one of the longest pipe lines in the world, second only to the 350-mile Kalgoorlie Goldfields Main in Western Australia.

The Tod River Water Scheme is designed to supply water to 10 000 sq. miles of agricultural land through a 240-mile main and 1 000 miles of distributaries. Water from the Tod River and its tributaries is diverted into a reservoir of approximately 10 000 acre-ft. capacity and thence pumped through 2½ miles of 18-in. rising main to a small concrete-lined reservoir on Knotts Hill, holding 20 acre-ft. From this reservoir the whole area is commanded by gravitation. The main varies from 20 to 10 in. and is of steel—the last 110 miles being concrete-lined. A break-pressure basin, also of 20 acre-ft. capacity, is provided at about 128 miles from the beginning of the main and 259 ft. below the full supply level of the Knotts Hill Reservoir. Rating is based on the unimproved valuation and varies from $0.08 (4d) to $0.14 (7d) per acre on all land within 1 mile of a main. The estimated consumption of water, based on actual figures in other areas is 132 gal. per acre, or 84 480 gal. (Imperial) per sq. mile. For the rate, 43 000 gal. are allowed and the excess is charged for at $0.49 (2s) per 1 000 gal. The total cost of the work will be approximately $14 100 000 (£3 000 000), or $6.00 (25s) per acre, but there is little prospect of the system paying more than operating expenses. The annual deficiency to be met by the State will be about $730 000 (£150 000), with no provision for depreciation. The work has greatly assisted in the development of the district and is, therefore, probably justified by its addition to the revenue and prosperity of the State.

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* *The Engineer*, August 10, 1928, Vol. CXLVI, p. 149.

The Kalgoorlie Goldfields Water Supply Project* (Western Australia) comprises a very bold and ambitious plan for supplying 5 000 000 Imperial gal. of water per day to the inland goldfields from the coastal ranges across more than 300 miles of arid and semi-arid country.

From the Mundaring Reservoir (capacity, 4 600 000 000 gal., or 18 400 acre-ft.), a series of eight pumping stations, at intervals, force the water through a 30-in. steel pipe line over a distance of 351.5 miles and deliver it at a level 1 290 ft. above the reservoir. The works were completed in 1903 at a cost of $16 000 000 (£3 250 000).

Since the decline of the demand for water for gold-mining purposes, a new and increasing demand has arisen for domestic and stock supplies to the splendid wheat lands adjoining the pipe line the development of which has been held up for lack of water. Not only does this demand absorb all the water thus made available but, in order to provide supplies for further areas, extensions to the system are now being considered.

Artificial Catchments.—The conservation of rain falling on roofs is as old as roofs themselves, but it is believed that the construction of large "roofs" on the ground—designed to catch the full water requirements of large farms—is quite novel.

There are parts of the Mallee country that are quite suitable for wheat growing, except that the ordinary channel system of water supply cannot be economically extended to them on account of the excessive cost of pumping, or of lining the channels. This is due to the porous nature of the soil, the absence of holding ground for the farmers' storages, or a combination of these factors. An experiment is being made to form a water supply of artificial catchments by laying sheets of thin galvanized iron directly on the ground discharging into water-tight covered storages.†

Taking the square mile, or 640 acres, as the unit for comparison: It may be assumed that a supply of water is needed for 6 persons, 10 horses, 2 cows, and 150 sheep. The daily consumption at maximum periods is established at 5 Imperial gal. per person, 25 gal. per horse, 8 gal. per cow, and 1½ gal. per sheep. Their requirements for a period of twelve months may be set down as 90 000 gal. net, or a little more than 500 cu. yd., the consumption being distributed throughout the year as shown in Table 3.

Examination of the available rainfall returns which cover a period of 50 years shows that 26 000 sq. ft. of a perfect catchment will provide the required volume in the worst period on record during that time, and naturally will yield considerably more in the great majority of years. Since the rainfall is not well distributed during the critical periods, a storage capacity of almost two-thirds of the minimum annual requirements is needed.

Experience has indicated that galvanized sheet iron has a considerable life in contact with the soil of the Mallee. Of the 26 000 sq. ft. of catchment, it is anticipated that the roof of the house and sheds will provide about 2 500 sq. ft. The storage tank itself, the volume of which has been fixed at 65 000 gal. to provide a safety margin, will have a roof area of 2 100 sq. ft., leaving 21 400 sq. ft. for the ironclad ground catchment. The tank is lined with cement concrete, 3 in. in thickness, coated with bitumen. An overflow channel leads surplus water away to prevent erosion. The tank is roofed with galvanized corrugated iron, 26 gauge, closely sealed all around and supported on 4 by $1\frac{3}{4}$-in. steel joists spanning the tank. An ordinary hand pump is fitted to the tank, which, together with the catchment, is fenced in with a stock-proof fence. If separate storage, about 7 000 gal., is provided at the house and sheds, the catchment tank may be reduced to 58 500 gal.

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* *Minutes of Proceedings*, Inst. C. E., Vol. CLXII and CCV.

† The method known locally as the "Ironclad Catchment" has been developed by Mr. A. S. Kenyon, an Engineer of the Victorian State Rivers and Water Supply Comm.

TABLE 3.—ESTIMATED WATER REQUIREMENTS FOR 6 PERSONS, 10 HORSES, 2 COWS, AND 150 SHEEP.

Month.	Gallons (Imperial).	Gallons per day.	Rainfall required on 26 000 sq. ft. (Points).
January	15 000	500	111
February	13 000	460	96
March	10 000	330	74
April	9 000	300	67
May	6 000	200	45
June	3 000	100	23
July	3 000	100	23
August	2 500	80	19
September	3 500	120	26
October	4 000	130	30
November	9 000	300	67
December	12 000	400	89
Total for year	90 000	250 (daily average)	670

To date (1929), only experimental public catchments have been constructed. For actual farm allotments it is proposed that the Commission supply the iron, cement, timber, and pump; pay the rail freight; land the bending and punching machines, and line and roof the storage tank. The settler is to cart all material to the site; prepare the catchment surface to receive the iron; bend and punch the iron; excavate for the storage tank, about 400 cu. yd.; cart the gravel from the railway (24 cu. yd.); and fence in the catchment and tank. If it is done in this manner the estimated cost is:

Catchment iron, 8 tons, and freight	$1 178	
Lining concrete tank 3 in. thick and coating twice with Colas, 284 sq. yd.	486	
Corrugated galvanized iron for roof, 1 ton	146	
Roof framing	97	
Fixing roof, pump, etc.	73	
Anchor timber nails, etc.	49	
Contingencies	306	
Total	$2 335	(£480)

This amount will be repaid by the settler in installments, together with his land purchase price. The settler will have the full responsibility for the management and maintenance of the catchment and tank, and no annual charge for that purpose will have to be met. He may, with profit, add both to the catchment and to the storage, as his circumstances permit.

WILLIAM E. RUDOLPH,* M. AM. SOC. C. E. (by letter).—Some notes have been contributed by Mr. East on the conservation and transportation of water for cultivation and stock-raising in the State of Victoria, Australia. These are particularly interesting because more than one-half the superficial area of Australia is arid, or semi-arid, a greater proportion than for any of the other continents.

It would appear from Mr. East's discussion that even the arid section of Northwestern Victoria (the Mallee Area) has far more rainfall than Northern Chile (three hundred times as much as in the nitrate pampa of the Atacama Desert). Evaporation is considerably less, ranging from one-half to one-third as great. Considering the scarcity of water, it follows that the costs of irrigation projects in Northern Chile would be much greater than those of the works mentioned as having been constructed in Northwestern Victoria during the past fifty years. These costs, in the writer's opinion, would be not less than $100 per acre developed—about one hundred times as much as the Wimmera-Mallee System described by Mr. East. Nevertheless, with the favorable progress now being made in the Chilean nitrate and copper industries located in the Atacama Desert, plans for making this region self-supporting are being widely discussed. The Chilean Government, in particular, has been seriously contemplating the fact that less than 10% of the country's area is productive, and according to a recent news note† it has allocated more than £4 000 000 for irrigation projects. One-half this sum is said to be intended for the construction of dams and canals to make use of the water of rivers in Northern Chile which at present are lost in the sands of the desert.

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* Chf. Engr., Green-Muth Bldg. Corporation, Rockville Center, N. Y.

† "Reclaiming the Chilean Nitrate Desert," *Engineering News-Record*, September 19, 1929, p. 448.

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Paper No. 1740

LETTING CONSTRUCTION WORK BY COMPETITIVE BIDDING*

By Edward W. Bush,† M. Am. Soc. C. E.

With Discussion by Messrs. L. C. Wason, Charles J. Bennett, Charles F. Chase, William E. Rudolph, Lazarus White, A. E. Waldron, A. Jordan Bernstein, Ernest Nicholson, Arthur W. Consoer, F. H. Stephenson, John C. Wait, Edward W. Stearns, Richard S. Kirby, J. E. Root, Spencer A. Snook, J. A. Lenecek, Jacob S. Langthorn, and Edward W. Bush.

Synopsis

This paper is an economic study of some of the successive steps taken in the preparation for, and the actual letting of, construction work by the method of competitive bidding. The parts or elements considered are:

A.—Preparation of the Contract, Specifications, Drawings, and Bond.
B.—Advertisement for Bidders.
C.—Information for Bidders, Questionnaires.
D.—Bidding Security.
E.—Awarding the Contract.

The endeavor is made to outline briefly a set of principles which, if followed, will assist in giving the owner the maximum value for the money expended in construction work and, at the same time, leave a profit for the contractor commensurate with the ability exercised in the performance of the contract and with the risk assumed in being bound to produce the work for the contract price. The owner supplies the funds, without which no construction work would be performed, and the contractor and his surety carry the

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† Engr., Aetna Casualty & Surety Co., Hartford, Conn.

inherent risks of the construction operations; it is uneconomical and against the best interests of all those engaged in the industry if any of these parties, or the engineer or architect, seeks to deny or withhold from any of the other parties the full value, protection, or profit that should rightfully accrue for his part in the undertaking. The owner, the contractor, the surety, and the engineer or architect are component parts more or less leaning for support on all the others. There is need of better harmony of effort. Perhaps this paper will lead to a better understanding among these parties.

INTRODUCTORY

In general, the paper supplements matter which has received the attention of many members in recent years.* Since 1917 the writer has reviewed a large number of contract documents, advertisements, bidding instructions, etc., on all kinds of construction work in all parts of the United States, and has been strongly impressed with the great diversity of methods in vogue on matters concerning which any group of engineers or architects should reach practically the same conclusions. Many of these procedures might affect the contract price to an amount of, say, 5 to 25%, or even more.

To illustrate: An engineer received bids on many millions of dollars worth of public work without even drafting a form of contract. The specifications and drawings were very sketchy, and the work was let to a company that tendered an informal bid on a new process which had never been tried on work of this size and character, and on which bids had not been requested. In reality, the contract was let without competition and all the formal bidders had their trouble for naught. The high character of the engineer precludes any collusion; but it was a queer way to expend the public's money.

In many cases the engineer and the owner word the contract so that the contractor becomes financier as well, because he must accept bonds or warrants at par in lieu of cash. Naturally, he must conclude a binding agreement with some strong financial party for the discount of the paper before he can prepare his bid. The owner may think he is borrowing the money at a nominal interest rate, whereas he may actually be paying a very much higher rate, but nobody but the contractor, his banker, and his surety knows it. Another variation of this is compelling the contractor to accept the unsold portion of securities issued to finance private construction. By this vicious practice many a contractor has been forced into bankruptcy as the market for the securities is saturated before he gets his securities and he cannot raise the funds needed to meet his expenses.

A certain architect prepares his contract, specifications, and drawings so that the bidders know exactly the amount and kind of work and materials required, hence the bids are always low and close. He gets the most for the owner's dollar. Another architect, doing the same class of public work, is known to furnish incomplete drawings and specifications and may change one without the other. They leave much to the imagination, hence bids vary greatly. Contractors who have done work under him are seldom the low bidders on new work. One such experienced contractor was the second bidder on a $1 500 000 job and $30 000 above the low bidder. The latter got the contract and collected more than $100 000 "extras" at completion. At times, it is doubtful whether this architect spends the owner's money economically.

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* The Society's Special Committees on Construction Contracts and on Engineering Contract Bonding have been engaged jointly with others in drafting the "Standard Contract for Engineering Construction," and the "Standard Questionnaires" used to determine the responsibility of a contractor, *Proceedings*, Am. Soc. C. E., March, 1926, Society Affairs, pp. 223 and 259, respectively; also, some of the members assisted the "Interdepartmental Board of Contracts and Adjustments," U. S. Bureau of the Budget, in preparing the standard contract forms now used by the United States Government; Frank T. Sheets, M. Am. Soc. C. E., has presented a paper on "Qualifications of Contractors on Public Works," *Proceedings*, Am. Soc. C. E., April 1928, Papers and Discussions, p. 1021; and Ward P. Christie, Assoc. M. Am. Soc. C. E., a paper entitled "General Contract System *versus* Segregated Contracts," see p. 98 of this volume of *Transactions*.

A.—PREPARATION OF THE CONTRACT, SPECIFICATIONS, DRAWINGS, AND BOND

Many contract forms give eloquent testimony to the fact that their authors never analyzed them. Inconsistencies abound largely because the same subject-matter is treated in several places. Many contracts are too verbose. Frequently, similar matter is given in the instructions to bidders, the contract, and the specifications. To express the same thought twice in the same language is useless repetition and to express it in different languages is to run the risk of confusion. Therefore, the following principle is evident:

(1) Treat any subject in only one place in the contract documents.

Nobody can see very far under water, or into the earth. As the owner may change his mind after the contract is signed, every contract should be drawn with the idea that several changes will probably be made. Changes lead to claims for extras; hence the document should be worded so that any change is immediately recognized by each party, and an understanding regarding it reached. Even after the contract has been signed, the parties remain "free agents" to set up a supplemental contract and thus many claims for extras are pressed. If a contractor presents a claim months after the work has been performed, submitting a mass of records favorable to his position, the owner and his engineer are at a disadvantage because they, not knowing of the claim, have failed to gather evidence on the case.

In one contract form used on a large volume of public work there are no clear-cut distinctions between (*a*) increase in quantities to be paid for at bid prices; (*b*) "cost-plus" work; and (*c*) additional work at a price to be agreed between the engineer and contractor. Probably the unwillingness of contractors to start trouble with those who can give them subsequent work, plus the fair-minded interpretations of the engineers, explain why trouble has not caused this form to be clarified. Hence, the principle:

(2) Recognize that changes are inevitable; provide a means for promptly coming to an agreement regarding them; and require that an "extra" must be promptly declared.

Whenever doubt exists as to the exact volume, kind of work, and materials, the contractor is apt to estimate the most expensive method or amount. He cannot afford to do otherwise. Any uncertainty in the contract leads to either a higher bid than is economic or to a claim for extras, depending on whether the contractor is called on to produce at a cost less or more than he figured on.

The phrase, "as directed by the engineer (or architect)", has cost owners a pretty penny in the past. A well thought out and detailed plan always indicates to the capable contractor that the engineer is competent and one with whom there is but little chance for disagreements. The next principle naturally follows:

(3) Every time an uncertainty is removed the price is lowered.

Many contractors are fully competent to supply an expert knowledge on certain construction matters far beyond the ordinary working knowledge of the engineer or architect. Often the results desired are specified instead of the sizes, methods, and materials; but it is unfair to hold the contractor responsible for both. To do so is to make him and his surety guarantee that the engineer or architect knows his business. The best current practice tends to be specific. This gives the contractor a more exact job to estimate, also places all bidders on an even competitive basis. Therefore:

(4) If possible, specify workmanship and materials instead of results, but never both.

Reasonable tests or guaranties that the owner is getting the results desired are proper when results are specified, but unfair if workmanship and materials are delineated. If the owner decides to use a special process or product, let him accept the guaranty of those promoting it instead of compelling the contractor to assume the responsibility as is so often done. Maintenance guaranties on roads and pavements are manifestly unfair because, assuming the workmanship and materials are all that they should be, no one can estimate correctly the volume and kind of traffic the road must carry later. After the contractor has completed the work in accordance with the terms of the contract his responsibility should end. Guaranties that run for a term of years increase the price and decrease the number of bidders as contractors do not wish to carry an open liability. Accordingly:

(5) In general, do not require guaranties from contractors where workmanship and materials are specified. If required where results are specified, limit the guaranty to those things directly under the control of the contractor.

Why ask the contractor to assume a risk that might better be taken by the owner? The financing of contracts, the rental of a suitable operating space, the depth of foundations that may be required, and many other matters affect the bid price. The engineer may have several months to study the proposed construction, whereas the contractor has only a relatively short time to decide what effect, if any, these things will have on the cost. In many cases, it will pay the owner to remove some of these uncertainties in advance of receiving bids. Therefore:

(6) It is uneconomical to hire a contractor to carry a risk which the owner can carry at a less cost.

Most State lien laws compel the owner to meet unpaid labor and material bills if the contractor is unable or unwilling to do so. A sub-contractor's bill falls in this same class. These lien laws vary among the several States. Many are grossly unfair and against the best interests of the construction industry, and hence a joint conference has been drafting a uniform statute. Lien laws are not enforceable against a State or the Government, so it has been the custom to give Government contractors the same protection they would obtain in private work, by inserting a clause that they shall pay these bills and also by stating in the bond that the surety is liable. Labor really does not need protection because it will not work unless payrolls are promptly met, and whether it is desirable to guarantee the bills of those supplying materials is open to question. This certainly makes it easy for the irresponsible contractor to buy materials and place sub-contracts.

In all States except one or two, contractors have the full right to take money paid on one contract to retire indebtedness incurred on another, to play the stock market, or otherwise to expend it—and some of them do so. In the few States which word the contract and bond liability so that the surety is not liable for unpaid labor and material bills, the cost of a contract bond is only about two-thirds of what it is in the States where the liability is covered by the bond, because the unpaid bills generally represent a considerable part of the loss assumed by a surety when a default occurs. Hence:

(7) On State and Government contracts, removing the surety's liability for unpaid labor and material bills, will undoubtedly lead to a reduction in the premium charge for bonds.

Plant or equipment (the words are used interchangeably) is something needed by the contractor for construction but not left in the completed structure. Usually it does not wear out on one job and frequently its cost when starting a new kind of work will equal a considerable percentage of the entire contract price. There is no reason why a public works commission should be more interested in seeing that the seller of the plant receives his pay than the seller of a piano, automobile, or anything else purchased by the contractor. Still, a form used in many important public contracts provides that the contractor must pay all equipment bills and that otherwise the surety is liable. In fact, many forms go so far as to state that if the contractor does not pay these bills the owner will do so and charge the expenditure as a payment on the contract. Thus, nothing may prevent the irresponsible bidder from getting all his bills for labor, material, and plant guaranteed. He trusts to luck that he will make a profit, but cannot lose much because he is not worth much. The best way is:

(8) Remove all protection to sellers of equipment, as this gives irresponsible contractors an unfair credit.

In many contract forms the drawings cannot be listed or identified, and many drawings lack a date and title which ties them to a specific contract. An individual, a co-partnership, and a corporation each must sign a contract in a different way, but many forms do not recognize this fact nor attempt to record the status of the party signing. Sometimes the contract price does not appear in the contract, but a reference is made to a bid having been tendered. Frequently, no mention is made as to which alternate bid is accepted. This lax way of letting construction work is not limited to that handled by inexperienced engineers and architects as much of it comes from supervising officers who know, or who should know, a better way. An engineer fails to discharge his full professional obligation, unless he sees that the contract form and drawings will cause his client, the owner, no embarrassment if a suit is brought. Hence:

(9) Prepare each contract with the idea that it may go into Court.

By a clause in most contracts, the contractor and his surety guarantee to save the owner harmless from all claims arising from the construction operations, patent infringements, etc. Many of these clauses are decidedly unfair because they try to make the contractor responsible for patent infringements on methods or material specified by the owner as well as on those for which the contractor himself is responsible. This leads to the principle:

(10) Do not word the "save the owner harmless" and "patent" clauses so broadly that the contractor must assume risks beyond his control.

There is a decided advantage in binding together the contract, general conditions, specifications, reduced scale copies of the drawings, bond form, etc., as many persons are properly interested in every construction work of any size, and should have the privilege of receiving the full information. Material men and sub-contractors should be bound to the main contractor with all the obligations that bind him to the owner; the terms of payment may affect the price bid by a sub-contractor; and the surety is interested in learning the exact conditions of all parts of the contract, specifications, and bond before agreeing to assume the suretyship. The best way is to give all of them the contract documents bound together; when separated, there is always the risk of submitting only a part of the proposition.

It is difficult to hold a sub-contractor or material man when he has made a losing contract, and he is apt to claim the contract voided if the method of making payments deviates in the slightest from what he had in mind when the agreement was set up. Details regarding the payments generally appear in the general conditions and these should always be attached to the specifications when copies of the latter are given out. Contractors experience grave financial difficulties at times because of having to pay 100% to material men while receiving only a partial payment from the owner covering the same items, and all because the material men bid on the specifications only, instead of the complete contract documents. It is not sufficient to state in the specifications that a certain standard form of contract and general conditions will be used, as it is always possible to insert a typed page in such a form; also, all such forms have places wherein written entries will be made. Therefore:

(11) It is desirable to bind in one document the contract, general conditions, specifications, and bond form, also small-scale drawings.

Constructions supervised by engineers are in general partly paid for monthly by unit prices applied after measuring the actual work performed during the preceding month. Constructions supervised by architects are generally paid for on a lump-sum basis and, as no accurate unit quantities have been computed, the matter of monthly payments is not so easily determined. Often the contractor submits a monthly statement of expenditures for work and materials which when approved by the architect becomes the basis for payment. This method works well if the contractor is making a profit, but if a loss will occur it may lead to the over-payment of the contractor, and the owner may be holding as retained percentage his own money instead of funds already earned by the contractor. A much better practice is to have the contractor as soon as the contract is signed submit a schedule of all the component parts of the work with prices attached, the total of which will equal the lump-sum contract price. Thus:

> (12) On all lump-sum contracts a quantity survey, with adequate prices attached, is needed to assure the correct computation of the partial payments.

The "retained percentage" is a fund earned by the contractor but withheld by the owner as a safeguard in case of default. For many years the established practice has been to keep back a certain percentage, say, 10%, of each amount earned. During recent years some contract forms are worded so that the percentage is only withheld from payments on the first half of the work. In such cases, just before completion the owner would hold about 5% of the total contract price as against about 10% under the older method. As a matter of fact, creditors push a contractor harder near the end of a large job (perhaps realizing that a loss is imminent) than during the earlier stages of the work when the contractor may be optimistic regarding a profit.

A bank that extends a large credit early in the operations will insist that the indebtedness be reduced as the work progresses and will compel the contractor to return a considerable amount from the monthly payments, or to assign payments. The bank is in a position to make a good guess as to conditions near the end of a contract and will always turn this knowledge to its own advantage. Most contractors on large work must make a considerable initial investment when starting a job before the payments are sufficient to carry the work, so the owner runs less risk in making liberal payments at the start than near the end of a job. During the time an existing contract is nearing completion a contractor may be just starting a new job, and this is another reason why the owner needs to retain the full percentage near the end of the work. To sum up,

> (13) More defaults occur near the end than near the beginning of a contract, therefore the owner needs to retain a greater reserve near the end than near the beginning of the work.

Any reputable surety company desires to be kept fully advised on the progress of the work covered by its contract bond. At times, it can exert a considerable influence on the contractor to keep him moving along to completion, and it will refuse to bond him on new work if the old work is going badly. It would serve the best interests of all concerned if the engineer or architect would promptly notify the home office of the surety of unsatisfactory progress or pending default. A good lawyer will always advise the owner to do so in the face of trouble, irrespective of any thing in the contract that indicates such a notice is not required. Especially is it desirable to obtain the surety's consent to any change in the method of payment or to any assignment of interest. Often it is difficult for the surety to obtain accurate information as to progress and this would be obviated if the engineer or architect would send to the Home Office of the surety a carbon copy of each monthly and final estimate. That is:

(14) Send to the home office of the surety a copy of each monthly estimate; also obtain from the surety its consent to any important change in the contract.

In most contract forms the authors fail to recognize that the "limits" of many kinds of insurance are just as important as the insurance itself. If a person using the streets is injured by an employee of a contractor very heavy damages might be recovered; also, the owner may be named as co-defendant with the contractor if a suit is started. Judgments for personal injuries are becoming larger and larger each year and the owner may need the protection of a policy having "limits" of $30 000 and $60 000, whereas the "limits" of the policy actually obtained may only be the minimum $5 000 and $10 000. Therefore:

(15) Carefully determine the limits as well as the kinds of insurance protection needed by the owner and contractor.

An engineer may have several months in which to prepare a job for competitive bidding, whereas the bidders have a comparatively short time in which to become familiar with the terms and details of the proposed work. The bidder adds nothing for contingency on an obscure point if there is none. On unit price contracts it is desirable to take up each item or classification and tell all about it; thus, the entire job is analyzed for the contractor and in such a way that he can price each item with the minimum effort. With this method the customary clause should be included stating that the contractor is obligated to produce a complete structure; and that the costs of parts, general expenses, etc., not specifically enumerated in the unit items, will be borne by the contractor and are to be included in the prices bid on the regular contract items. The method will also prevent disputes on wrong classification during the construction period. Hence:

(16) On unit price contracts separately consider each bid item in sequence, telling where it is found, the amount at each place, with complete details.

If the drawings are not clear, the bidder cannot make a close estimate; also, there is a greater chance for disputes. In many cases more and better drawings will lead to lower bids. Isometric perspectives will often add considerably to the ease with which the entire project is assimilated by the bidder, and by the superintendent who later builds it. In other words,

(17) A little extra effort on the drawings is generally worth more than the cost.

The simplest form of bond is that guaranteeing the completion of the work in accordance with the terms of the contract, also saving the owner harmless against liens, claims, loss, or damage due to causes within the control of the contractor. This will give the owner all the protection he needs. The surety companies, however, seldom draft the bonds and frequently a form is presented that greatly increases the hazards of suretyship. Some forms double the premium charge without adding any benefit to the owner. The surety company always wants to know the exact wording of the bond before agreeing to assume the suretyship, and it is in the interests of conservative underwriting that the sureties should obtain this information. One standard form of contract states that the bond required will be "in such form as the owner may prescribe" and the surety is left in ignorance of what will later be presented. Through ignorance a "forfeiture" bond might be drafted where a simple bond form was intended.

A certain extensively used bond form states that the contractor shall "satisfy all claims and demands incurred for the same". If these words were followed by the phrase "for which the owner may be held liable", the liability would be understandable; or if this extension of liability were worded instead "and pay all bills for labor and materials used on the work", a definite suretyship liability would be created. It is doubtful whether the person who drafted this form really had a clear idea of what he was doing. The bond is really paid for by the owner and it should be drawn so that he receives full protection and no one else. For many years a certain State has saved just one-half on its premium charges merely by adopting the proper bond form. The best practice, therefore, is to:

> (18) Include the bond form with the papers given to the prospective bidder, and word it so that only the owner who pays for it will receive the protection bought.

The introductory part of many contract forms defines words like "guaranty", "surety bond", etc., and elsewhere in the contract are clauses describing the obligation of suretyship. The bond form itself generally describes this obligation. In reviewing a large number of contract documents the writer has noted, almost without exception, that the different descriptions of the obligation always disagree. From time immemorial bonds have been worded in a peculiar negative manner, and this language is not the kind that is readily inserted in a contract. Hence:

> (19) The place to delineate the obligation of suretyship is in the bond and not in the contract.

While some financially strong contractors are able to obtain a fair volume of work without competition and at attractive prices (all contractors hope some day to be in this class), most work is contracted after competitive bidding has brought the price down. The plan works best when a surety bond is taken for the owner's protection. This is generally required when public money is spent, and the volume of bond protection on private work is constantly increasing.

When a surety issues a contract bond for a contractor it extends a credit to him because it increases his financial strength for the benefit of the owner in relation to the work bonded by an amount equal to the penal sum of the bond. A contract bond does not guarantee the owner that the contractor is responsible although many of the criticisms aimed at the sureties in recent years have assumed this. The obligation of suretyship is fully discharged by a monetary payment made to the owner after he has suffered a loss caused by the contractor failing to discharge his contractual obligations. It is necessary, however, for the surety to determine the contractor's responsibility for its own protection before assuming the contract bond obligation. Failing to do this, the surety will suffer the inevitable losses that go with careless underwriting. Others may form an opinion on the responsibility of a contractor, but they do not back the opinion for the benefit of the owner as does the surety company when it writes a contract bond.

Many times during recent years the surety companies have been told that they should render an inspection service along with the bond protection so that the owner will receive the surety's guarantee that the work is being performed strictly in accordance with the plans and specifications. This would trespass on the professional services of the engineer or architect and is not a function of suretyship. It is an unworkable scheme.

Surety companies are constantly dealing with claim cases and their investigations generally disclose the causes of failures. Strange as it may seem, the inexperience of the contractor and the inadequacy of the contract price, while important elements, are not the principal causes of defaults, as the sureties pay many of their largest losses on contractors who are experienced and apparently were very well qualified when the bonds were written. Over-extension, or taking on more work than is warranted by the contractor's organization, plant, and financial strength, is probably the greatest single cause of failure. Many contractors plunge on real estate and otherwise tie up their capital in speculative investments where it will not work for them when needed to turn a financial corner. The defaults of important subcontractors or failure to get labor and materials for the prices estimated, have caused many contractors to fail. It has even been stated by some contractors that the high cost of educating young engineers and architects in the technique of construction work has kept many from their estimated profits. There are many ways in which a contractor becomes irresponsible after the contract is signed, and, realizing this, the surety companies when re-letting defaulted contracts almost invariably require for their protection contract bonds executed by other sureties. They know the protection is worth more than the cost of the bond. It is a fact, that:

> (20) The owner will generally receive the maximum value for his expenditure by letting work after fair competition and requiring a contract bond for his protection.

B.—Advertisement for Bidders

The purpose of the advertisement is to attract bidders. Therefore, it should be worded so as to show the prospective bidder whether or not he should spend the time and money necessary to investigate the project. A glance at the advertisements in any technical publication will prove that many of them fail to meet this requirement. A contractor will probably make a lower bid when the job fits his organization and plant because he knows just what the costs have been on similar work, and he may have just the plant needed. It is expensive to investigate prospective work, and this "over-head" is only paid for from the jobs that are obtained. Anything that will reduce the "over-head" benefits the industry.

Recently, an engineering paper carried two advertisements side by side for bidders on public work. One briefly gave the location, described the work, and the amount of each kind to be done; also it mentioned that the details could be obtained at the site (which was remote from the head office of the organization letting the work) as well as at two other locations convenient to bidders. The approximate bid price could easily be computed and the prospective bidder would know at once whether he should investigate. The other job was at least two days' journey from the office where the details could be obtained after depositing $25; no plans were available at or near the site, and it is thought that most of the prospective bidders were remote from where the plans could be obtained. Whether the job would amount to $100 000 or $1 000 000, was left to the imagination. While the advertisement did give the date on which the bids would be received, it did not tell the place, whether at the site or at the distant main office.

After spending some time and money investigating a prospective contract, the contractor is apt to put in a bid, anyway, even although the job is larger than he can finance; therefore, the advertisement should indicate the probable contract price. The surety companies also would greatly appreciate this information because nearly every experienced contractor approaches his surety to ascertain whether a performance bond could be obtained, before putting in a bid on a large contract; and often the surety is asked to pass on the matter before the contractor is in a position to make his own approximate estimate.

The surety companies subscribe to engineering publications largely for the advance information to be found in the advertisements to bidders, and telegraphic submissions of bond cases often refer the surety to certain publications for details. Every time a surety authorizes a bond that should not have been authorized or fails to authorize one that should be written, the best interests of all concerned suffer.

The following form of advertisement is suggested:

........(owner)........ will receive, publicly open and read, bids at its office at ....(address)........ at ....(hour)...., ...(date)......., for the construction of a ...., more fully described below.

(Here insert a paragraph giving location, accessibility (if desirable), and description in narrative or schedule form of the principal parts, items, or quantities. Include any pertinent facts regarding local conditions, the type of construction, the materials to be used, the size of buildings, etc., that will assist the contractor in determining whether his previous experience, plant, organization, and financial strength makes the work attractive to him.)

Approximate estimate of bid price between $............ and $.........., certified check or bid bond of $..........., required, and performance bond will be $........... (or ....% of contract price). After ......(date) ...... the plans, contract, and bidding forms can be obtained from above mentioned office, or from.....................................

(Dated).................... (Signed)........................

The following principle holds:

(21) It is to the owner's advantage to word the advertisement so the bidder can tell whether the proposed contract will fit his ability, experience, plant, organization, and financial strength.

C.—Information for Bidders, Questionnaires

In the "Information for Bidders" is the place to state that the right is reserved to reject any or all bids and to tell the bidder where and how the bids will be received as well as what evidence, if any, he must present to qualify for the contract, should it be awarded to him. The contract later to be set up is supposed to be a fair "meeting of the minds"; hence, anything told the bidder in the "Information" that influences his conception of the proposed work, has much the same virtue as if included in the contract. It is believed the Courts have generally held this view.

Certain clauses in the general conditions of the contract, as on the bidding security, time allowed for signing the contract, etc., could with propriety be transferred to the "Information for Bidders", thus properly confining the contract proper to matter describing a method, detail, part, or condition of the consideration that one party gives the other. Hence:

> (22) Many contract forms would be improved if clauses on subjects which are active before the contract is signed were placed in the "Information for Bidders" instead of in the contract.

Every prospective bidder should have equal privileges in obtaining information about local conditions and whether or not alternate or informal bids will be entertained by the owner. It is decidedly unfair to let formal bidders expend their time and money preparing bids and then to award the contract to some informal bidder. If the owner is dissatisfied with the method he has specified and wishes to consider another one, the fair thing is to reject all bids or postpone the letting until all bidders can submit amended tenders for the work. That is:

> (23) Each prospective bidder should receive all the information given to any other bidder; and have the same opportunity to submit alternate or informal bids.

One of the best ways to minimize over-extension would be to give the prospective bidder a chance to limit the volume of new work he could receive at any letting, provided bids are taken on more than one job. The owner thus secures more competition per job and loses no economic advantage. The contractor is benefited because he has a better chance of getting a job when bidding on a large volume of work than on only the additional quantity that will fit his organization and ability to finance. Therefore:

> (24) Permit bidders to limit the amount of new work that may be awarded to them at any letting.

Most constructions supervised by architects are built amid urban surroundings where materials like sand and crushed stone are staple commodities with fixed prices. Many jobs supervised by engineers are at remote places where even the water supply may influence the cost. All specifications call for "clean and sharp" sand; but many localities cannot produce such a material although the local sand has been used satisfactorily. The engineer will secure much lower bids if all local sources of materials are investigated in advance. The writer recalls a case where it is thought an advance expenditure of $500 by the engineers would have saved a very large sum, possibly several hundred thousand dollars. Principle (3) is also pertinent to this subject. The best plan, then, is to:

> (25) Investigate local facilities and materials and furnish the information to all bidders.

Among contractors, engineers, and architects, there is a tacit understanding that the owner will proceed with the award within a reasonable time after receiving bids; but every now and then a contractor finds himself the low bidder with no definite idea of when, if ever, the contract will be signed. Such a delay is unfair as he hesitates to bid on other work; also his bidding security is still in the hands of the owner, and it may be difficult or impossible to obtain additional credit with which to bid on other contracts. As long as the bid is outstanding it is a potential contract and it affects his "responsibility" for other work. It is only fair, therefore, to:

> (26) Specify that within thirty days after receiving bids, the award will be made or all bids will be rejected.

In a few States the statutes require that the engineer's estimate of quantities and bid prices on highway work be made public in advance of the receipt of bids. This is against the best interests of the States, as it makes it too easy for inexperienced contractors to prepare bids. What may be a fair price for an experienced contractor with full equipment and a good organization may be wholly inadequate for a beginner. If the bidder is qualified to construct the work, he is qualified to prepare his own estimate. It is proper and desirable to tell the prospective bidder the approximate bid price within limits, but not to give him a detailed estimate or to ask him to bid on a percentage of the engineer's estimate as is often done. Therefore:

> (27) Do not give prospective bidders detailed estimates of proper bid prices on road or similar work.

For building work, where lump-sum bids are generally required, bidders must estimate the quantity of each item before they can fix the price. This is a laborious operation on a large building, and the cost is added to the bid price. Why should the owner pay the expense of ten, or more, quantity surveys on one job when he can provide one survey to serve all bidders, and at the same time bring all bidders into true competition on the basis of price only? In many localities groups of contractors find many advantages in having joint quantity surveys made. Any such plan requires distinct drawings and specifications, as uncertainties are bound to be uncovered by those preparing the quantity surveys. Furthermore, corrections made before the work is offered for bidding, lead to lower bids. As a general principle:

> (28) It is economic for owners to offer quantity surveys to bidders on building work.

Many advertisements for bidders are worded as if the owner were conferring a great favor on the contractor to permit him to bid—in fact, some owners charge the contractor a good fee for the plans. In reality, the favor is extended by the bidders because it costs more to prepare the bids than to provide contract documents. After a bidder has qualified it is to the owner's advantage to loan him without charge all the contract documents he really needs in order to prepare an estimate carefully. Making it easy and inexpensive to bid is bound to be reflected favorably in the price and a $100 expenditure by the owner for additional sets may return him thousands of dollars when the bids come in. Many parties as sub-contractors, material men, and sureties have a proper interest in reviewing the contract documents carefully before the bid is tendered.

Frequently, when deposits are taken, the time within which the plans must be returned is given as seven days after bids are received, and some advertisements state that all the documents must be returned with the bid. This is unfair, especially to parties who may have given sub-bids. In one metropolitan district sub-contractors on a certain class of work had so many disputes with contractors that they announced a decision to retain the drawings and specifications until such time as their bids had been definitely rejected. They had been asked too many times to perform at the old prices after changes had been made by the owners subsequent to the receipt of bids. Until all bids are rejected or some bid is accepted, the matter is open; therefore, the return of plans should be based on a period starting from the award of the contract or the rejection of all bids instead of from the date bids are received. A 14-day period is suggested as a fair condition. To sum up:

(29) Be liberal rather than close as to the number of sets of the contract documents given to prospective bidders. Do not charge for these, nor ask for their return in less than fourteen days after the award or rejection of all bids.

Some contractors, apparently, get a great deal of satisfaction in bidding high without any desire to receive the award; others put in high bids merely because the engineer or architect asks them to estimate the jobs and they desire to remain in his good favor. Still others, having employed estimators, think they must keep them busy. A complimentary bid is disturbing and serves no real purpose. Often it causes difficulty for the low bidder to obtain a bond, or credit from his bank, because a few very high complimentary bids from contractors who are known to have a good knowledge of costs, make it appear as if the low bidder had greatly under-estimated the job and, therefore, will have a losing contract. Hence:

(30) Engineers and architects should not encourage contractors to submit complimentary bids.

Available local facilities have a considerable influence on the prices which must be paid for certain constructions. Engineers have been known to increase the prices on their own contracts by letting so much work within a short time that there is no real competition as each contractor having the requisite plant is reasonably sure of obtaining all the work he desires. A certain owner always tried to spend the large sum available at a rate about 50% faster than the contractors could work, due to limited facilities for labor, materials, and transportation. As a result bid prices constantly rose although the contractors failed to make profits on most of the work. A large number defaulted and others withdrew after suffering large losses. In other words:

(31) To obtain economic bids, do not let more work than can be readily performed.

The allowance of the customary week or ten days after the award within which the contract must be signed and the bond given is often much too short and fourteen days should be allowed instead. The surety companies are hard pressed at times to underwrite bond cases, secure re-insurance from other companies, etc., within the short time available. With the extra days a case could be presented to the surety by mail with all the supporting documentary evidence, giving the underwriter a clear-cut picture, instead of rushing the case through by telegraphic presentation. It is on the side of conservative contract bond underwriting and does not really delay the actual beginning of the work because the contractor starts on his preliminary arrangements just as soon as he receives the award. Therefore:

(32) Allow fourteen days after the award for signing the contract and giving bond.

A commendable practice is growing; the prospective bidder is required to fill in a questionnaire form demonstrating his responsibility before he receives the bidding forms or is awarded the contract. It is unfair to encourage him to bid and perhaps later deny him the award, if low; also it is thought that a contractor who has too much work on hand to warrant his being allowed to bid will make less disturbance if disqualified before bidding than after finding himself the low bidder. The forms are still quite new and undoubtedly changes will be introduced from time to time. Few of them direct an inquiry regarding outstanding bids or the amount and status of other contracts on hand, and this is a serious omission because the volume of work is an element which enters into any determination of responsibility.

One form asks the bidder to describe the methods or plant he will use if awarded the contract. This is an unfair question to ask of an experienced contractor. If the engineer wants a certain kind of plant used he can specify it, but if the contractor has to produce a specified volume and quality of construction work he should be allowed the greatest latitude in the selection of methods or plant. The construction industry owes much to the pioneers who have dared to use new processes and thus have found a cheaper and perhaps a better way of producing an old form of work. Why should the contractor disclose his methods before the contract has been awarded to him, especially as all bids may be thrown out and his competitors may consider the new method at the re-letting? In short:

(33) Use questionnaire forms to qualify prospective bidders instead of to justify awards.

D.—BIDDING SECURITY

Bidding security is not generally required on private construction work; but on public work usually a deposit with the bid of either cash, Government bonds, a certified check or cashier's check, or a bid bond is required. At times, the bid must be accompanied by a letter or certificate issued by a corporate surety company stating that it will write the contract bond for the bidder should he be awarded the contract. The bidding security is supposed to be a guarantee deposited with the owner binding the bidder to execute the contract and file the required contract bond should the contract be awarded to him.

References to the bidding security are generally included in the advertisement and "Information for Bidders". In many proposal forms the clause stating the conditions under which the bidding security is deposited as a guaranty fund for the owner's protection is decidedly unfair because it provides that, if the contractor fails to execute the contract and file the bond after an award, the fund is forfeited to the owner. The measure of damage to the owner is the difference between the bid that was not executed and that of the next, or some other bidder, who qualified. The owner after thinking so well of the contractor as to encourage him to prepare and tender a bid is treating him rather harshly to turn any possible misfortune to his own advantage.

The amount of the bidding security is generally about 10 or 15% of the contract price. It is desirable that the owner state a definite amount rather than a certain percentage of the bid price, because the bidder may not know what he will bid until just before the bid is tendered. The practice of some public work bodies seems to encourage the financially weak contractor to bid on work beyond his capacity as they graduate the size of the security required up to a maximum of, say $25 000, which sum would apply to any contract of more than a certain amount. In one case the maximum security of $15 000 was supposed to give the owner protection on a contract of nearly $500 000. A $50 000 bidding security would have been much more appropriate. It is best, therefore, to:

(34) Designate a lump sum for the bidding security in an amount from 10 to 15% of the probable contract price, and condition its receipt so that the measure of damage to the owner, if the bidder fails to execute the contract and file a bond, is the difference between the amount bid and the amount for which the owner may be able to award the contract within a reasonable time.

Many engineers, contractors, and surety companies believe it is desirable to abolish the use of bid bonds and hold strictly to the use of cash, Government bonds, or certified checks. The requirement that checks shall be filed is increasing and bid bonds may soon be a thing of the past. Almost all bidding checks are borrowed from a bank because they may be on deposit for some time after the bids are opened, and contractors seldom have enough surplus cash to permit a considerable sum to be taken from the business. This borrowing is really desirable as it brings them together at a time when the banker may give the contractor some good advice against taking on more work than can be easily financed. Many contractors after such talks have wisely decided not to bid. In all cases, then:

(35) Do not permit bid bonds as bidding security.

The letter issued by a surety company stating that it will write the contract bond if the bidder is awarded the contract is called a "bid letter" by the surety, and is handled by it just the same as a bid bond. All the reasons that make bid bonds objectionable apply equally to bid letters. The owner fixes the amount of the certified check and should make it large enough to supply the protection desired without trying to bring in a surety company to bear part of the load. It is in line with conservative underwriting if the surety gives no letter or certificate to the owner when bids are received, because seldom has the surety had time to give the case the full investigation needed. Therefore:

(36) Do not ask for bid letters.

E.—Awarding the Contract

Often bids have been prepared at considerable expense, and it is not treating the bidders fairly unless they are accorded the privilege of being present when the bids are publicly opened and read, as this is the only guaranty that there is real competition. Otherwise, the bidder is apt to suspect that the job is "set" for some favored contractor. Public opening is almost universal when public money is to be expended, but on private work there is still a considerable proportion of secret openings. It is claimed that often after a secret opening the better class of contractors are asked to cut their bids to meet the figures of irresponsible contractors who have nothing to lose and who therefore bid much lower than good construction work can be produced, and who may have been asked to bid for this reason. So flagrant has this practice become in certain localities that the reputable contractors have found it to their advantage to file duplicate bids, one with the owner and the other with some designated person who later gives each bidder a tabulation of the bids. Naturally, the owner does not get very far in "peddling bids" when the best contractors know all the details. So it is good policy that:

(37) Bids on public or private work should be publicly opened and read.

Scarcely any reputable contractors are averse to bidding on important contracts located in other places. It is generally considered strictly proper for contractors to bid on distant work and for owners to invite such bids. Frequently, however, an out-of-town bidder is low and a local contractor is next; then the greatest pressure is brought to bear on the owner to let the work to the local bidder under the plea that "all the money should be kept at home". Such an award would be unfair to the outside bidder who was encouraged to bid and who believed, tacitly perhaps, that if he was low, he would receive the award. If only local bidders are desired, the advertisement should state this condition. Even should the out-of-town contractor do the work, most of the money will stay at home because it is largely the local materials that will be used, also local labor; and the contractor only takes away with him the profit, if any, at completion. If he loses because he bid under the local bidder, the out-of-town contractor might, in fact, make a substantial contribution to the community. The rule should be:

(38) Award the contract to the lowest bidder if he is qualified, irrespective of what a local bidder has bid.

No owner wants his work performed by an irresponsible contractor, nor do engineers desire him to undertake contracts supervised by them. It is supposed to be the professional duty of an engineer to advise the owner regarding the low bidder and to urge the award to a responsible contractor.

Any contractor who is dishonest, tricky, and bears a bad reputation with those for whom he has done work is, of course, irresponsible no matter how many ill-gotten gains he possesses. Fortunately, there are not many such in the construction industry, and the contractors who are doing the greater part of the large yearly construction programs are men with just as high ideals and character as will be found in any other large industry.

However, merely being honest does not make a contractor "responsible" to undertake a large contract. In addition he should have ability as evidenced by his past experience and record; an organization suitable for the proposed work; and sufficient working capital or net quick assets to finance the operations and reasonable additional costs. These qualifications should give a fair assurance to the owner (or a surety company) that the contractor is well able to complete satisfactorily (*a*) all the unfinished contracts on hand; (*b*) the new contract in question; and (*c*) all the contracts subsequently taken until the new contract has been completed and all bids have been paid. When a contractor gets into trouble on one job it soon spreads to all his contracts; therefore, the entire volume of work he is obligated to perform is a very important element in determining his responsibility, although many do not give this element due weight. Assuming the contractor is otherwise qualified, his responsibility is determined by the ratio his financial strength bears to the total volume of work he must perform.

When the contractor is working well within the proper ratio, he should be a good risk for himself, as he will probably have sufficient assets to meet an ordinary financial strain if necessary, and so will continue in business. When he is over-extended or suffers bad luck, he may be forced into bankruptcy because credit quickly becomes afraid and runs away in the face of trouble—that is, all credit except the bonding credit which has to stay because a contract bond is a non-cancellable obligation. While the ratio of the financial worth to the total work should always be determined, so that it can be duly considered when judging the responsibility of a contractor, it is doubtful if any fixed ratio rule could be applied generally, because contractors vary in their ability to overcome difficulties and because many kinds of work are inherently much more hazardous than others. Probably no two persons will reach the same conclusion on responsibility, although experienced surety company underwriters will frequently agree on how much work of a certain class a contractor should undertake at any one time.

Many surety companies have a basic ratio rule about as follows: Assuming the contractor is capable, experienced, and has all the plant and organization needed, he should have unquestioned net quick assets of not less than 10% and other assets of an additional 10% of the total volume of work to be performed. Unusual ability and a kind of work that is not hazardous might lead the surety to reduce this ratio, while work of a hazardous kind, or under unfavorable local conditions, would lead the surety to desire more working capital in proportion to the volume of work.

Many engineers, architects, and contractors do not recognize any ratio rule in determining responsibility, but classify a contractor according to the size of the contracts previously performed by him without any consideration of the number he is qualified to handle and finance at any one time. A contractor who has a large plant and little working capital is a poor risk, even on a contract that is well inside the ratio, and is taken when he has no other work on hand, because, in all probability, he will soon take on other contracts in order to keep all his plant at work and then will have too much under way at one time. Too much plant without corresponding net quick assets has wrecked many contractors, especially in the road-building business. A contractor can often make more money closely supervising one contract than in trying to handle several, some perhaps at a distance, and this has been proved over and over again in the financial statements filed with surety companies.

While it is comparatively easy for an engineer to follow the progress being made on a contractor's work on hand, it is not so easy to analyze his financial statement and determine what allowances should be considered in arriving at the net quick assets and net worth. All parties habitually dealing with credits will disallow or greatly scale down all obscure items; contractors, unfortunately, have no standard method of keeping their accounts or preparing financial statements, so that many asset items cannot be evaluated without the supporting information.

Anticipated profits are frequently included, and many contractors think every dollar expended on a contract is bound to come back accompanied by the estimated profit. Notes receivable may have been given in payment for a junior interest in the co-partnership or corporation which conducts the business, and these really add nothing that has a quick sale value. Stocks and bonds may appear at inflated values. The contractor or a certified public accountant can say under oath that the statement shows the true condition of the books, but such an oath does not necessarily determine what net quick assets or net worth should be allowed. All these considerations lead to the principle:

> (39) Ascertain the ratio between the contractor's financial strength and the volume of work he is obligated to perform. Use this as an aid in determining his responsibility before awarding the contract.

DISCUSSION

L. C. Wason,* M. Am. Soc. C. E. (by letter).—This paper is eminently fair, comprehensive, and complete, and the writer endorses each of the thirty-nine recommendations, except in a few cases, as follows.

The author calls for small-scale drawings, Principle (11), but the writer thinks that these would be expensive and unsatisfactory, while a photostatic reproduction of the contract plans would be simple and inexpensive.

The Aberthaw Company has sometimes followed the method suggested in Principle (12), but has found it somewhat unsatisfactory because the engineer or architect has used these items for purposes of adjusting or computing extras when they were not compiled for such a purpose. When compiled for payment purposes only, they are sometimes unbalanced for one reason or another, and, therefore, if used for another purpose, such as computing extras, they may do a rank injustice.

To overcome this objection the Company has followed the practice of making payments at a certain point of progress which could be easily determined as, for instance, on a building: Completion of foundation, the completion of first floor, second floor, etc., the roof, and other progress points in regard to interior finish and completion. This method has been satisfactory to both the engineer and the contractor.

The writer agrees with the principle stated in Principle (13), but past practice has permitted the owner to withhold very large reserves, much more than are needed to protect him, and has tied up a great deal of the contractor's capital, whereas if the owner had retained only enough to protect himself and had paid the contractor the remainder it would have enabled him in many cases to complete his contract without a default, because he would then have enough cash with which to carry on his work and to pay outstanding bills. A clause should be added to the effect that only enough reserve be retained by the owner for self-protection and all the surplus be turned over to the contractor.

The writer agrees with the principle stated in Principle (26), but would cut the time to ten days. The owner knows the maximum amount he is willing to put into the undertaking, and ten days is ample time to discuss and digest the bids received and to determine whether he wants to go ahead.

As for the principle stated in Principle (27), the writer suggests that the method for the engineer to follow is to read his own estimate into the record of the bids when opened. This is being suggested as a method for the Federal engineers when they receive bids and plan on doing the work by the day as they have on a number of occasions. Then, of course, a direct comparison is available between the engineer's estimate and contractors' bids, and by this method is not incurred the objection which Mr. Bush suggests, with which the writer agrees.

As a whole, Mr. Bush has made a very comprehensive and important contribution to the relationship of engineers and architects with contractors, which should have wide circulation and careful reading.

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* Pres. and Treas., Aberthaw Co., Boston, Mass.

CHARLES J. BENNETT,* M. AM. SOC. C. E. (by letter).—The author apparently is familiar with the letting of contracts. He approaches the subject from the standpoint of an engineer's interest in securing satisfactory contracts and resultant excellence in work. His viewpoint is that of the surety company. Nevertheless, the paper presents a broad discussion on the subject of contracts and specifications and is worthy of careful perusal by engineers. Attention is called to the necessity for clearness and continuity in writing specifications and including in the contract form carefully written clauses covering the character of the work to be done and the conditions specified. Experience in letting a large number of contracts convinces the writer that much work is necessary before engineers and architects come to the point where the contract and specifications will be sufficiently clear to advise the prospective bidder just what he has to do. The paper gives much information which will aid in securing this desirable result.

The writer is convinced that the average specifications and contract fall far short of achieving the ideal. Particular attention is called to the fact that ambiguous clauses may be interpreted by the engineer to the disadvantage of the contractor and in some cases at the expense of the owner. Emphasis must be laid on the necessity for drawing each contract and specification carefully to cover the particular work in hand. In other words, no attempt to apply a standard specification to a particular job will be successful. It seems trite to state, but it is nevertheless true, that the greater the care in preparing these documents the better the results in securing a reasonable cost and excellent work. The author discusses thirty-nine topics all of which are valuable in studying the question of contracts and specifications. Difference of opinion may arise on some of these principles, but, in general, the paper is sound and reasonable and merits careful reading by engineers.

The writer does not entirely agree with the author regarding the removal of the surety's liability (Principle (7)). His experience has been that often labor is not paid regularly and that protection to this particular item in the cost of construction should be included in the liability bond. The writer is willing to question also the statement that protection to sellers of equipment (Principle (8)) should be removed and would qualify it by saying that that portion of equipment bills which is properly chargeable to the work in hand should be protected by the surety. He is quite in accord with the author, in the elimination of guaranties for maintenance.

CHARLES F. CHASE,† M. AM. SOC. C. E. (by letter).—The first part of this paper interests the writer particularly because it emphasizes some matters on which the American Institute of Steel Construction as well as the Structural Steel Board of Trade of New England and other organizations have been working. The point is that so many architects and even engineers leave it to a contractor or bidder to guess what may be required when the specifications contain such clauses as:

> "Furnish all steel, or other materials, whether indicated by the drawings or specifications or clearly required to carry out the intent of the design."

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* Cons. Engr., Hartford, Conn.

† Pres., The Berlin Constr. Co., Inc., Berlin, Conn.

The architect or engineer is supposed to be paid by the owner to make plans and specifications that will give the owner what he wants and to give a contractor complete information as to the requirements. If such plans and specifications are not complete and definite there are as many guessers as there are bidders and, in many cases, the lowest bidder or guesser, if he secures the contract, has undertaken something which will make trouble for him from the start. If all uncertainties are removed the owner will be benefited, for he will pay no more (probably less in the end) and will get what he wants.

The writer cannot understand why so much is left to the imagination of a steel fabricator. The same thing applies to all construction and from personal experience, the writer knows that many contractors base their bids on what they know of the architect or engineer as well as on the plans he prepares.

The matter of contractors' bonds is also a very vital one, and all sub-contractors are interested in what protection they get if the general contractor defaults. Many general contractors assume obligations which they are unable to finance. The result is that sub-contractors have to help carry the work and this can be done only at increased cost to themselves, or, in case of a default, they are the greatest proportional losers, unless the surety company is bound to pay them in full. Of course, if the surety company assumes this obligation, the premium has to be made commensurate with the risk and the public, in the end, pays the bill.

William E. Rudolph,* M. Am. Soc. C. E. (by letter).—The author mentions the unfair "peddling" of bids. This practice does not appear to be confined to instances where bids are opened behind closed doors. It is now quite common, after a public opening, to have estimates re-compiled by three or more of the low bidders under the pretense of revision in plans and specifications, although the changes may be of distinctly minor importance. Naturally, each of the "favored" contractors knows what his competitors have bid and what figure he must submit if he wishes to get the work. The result too often entails loss for all parties concerned. When minor changes in plans or specifications are to be made, such as to bring the cost of a job within an appropriation, it would appear a fair rule to do business only with the lowest qualified bidder.

With reference to the economy of furnishing quantity estimates to bidders, contractors are being circularized by estimating agencies which offer to sell such estimates on any particular job. How reliable many of these estimates may be, is an open question. Their sponsors must be receiving some return for their efforts or the practice would not continue. Certainly the engineer or architect who prepared the plans is better qualified to furnish such quantities than unknown outsiders.

In support of Mr. Bush's Principle (29), there appears to be no justification for requiring a contractor to return his set of plans and specifications with his bid. What other proof has he of the amount of work which he has bound himself to perform?

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* Chf. Engr., Green-Muth Bldg. Corp., Rockville Center, N. Y.

Lazarus White,* M. Am. Soc. C. E.—This is a very valuable paper, because the contract under which one works, either as engineer or contractor, is perhaps the principal instrument in the accomplishment of that work. The relation of the engineer to the contractor itself contains the human element, and as no contract ever automatically executes itself without the aid of human beings, one should consider the contractor, in this relation, as a human being entitled to justice. He takes all the responsibility and risks his capital and reputation on the successful performance of the work, and in that he is perhaps aided by the surety company.

The most ancient definition of a contract is the clearest—a contract is "a meeting of the minds". One party wants something done, and he asks the other party, "How much will you do it for? Here are the conditions". What does that imply? Does it imply that the first party has the option of saying "even if the conditions vary—no matter how much from the conditions described—the price still holds"? That is practically what all contracts endeavor to do. Instead, the contract should state:

> "These are the conditions, and if done under these conditions this price will hold; if the conditions are found to be different, then we will make necessary modifications and adjust the price."

The speaker happens to know of cases where that has been done, and the result was remarkably cheap work. The contractors assumed that the conditions described would be met, and they knew that the engineer with a strong sense of justice, would modify the prices if conditions were found to be different. If a contract is based on certain stated conditions, and those conditions are found to be different, the price is modified so that the contractor "comes out whole".

In city contracts elaborate borings are sometimes made; but despite that all the information is covered with waivers, practically admitting inaccuracies; and an attempt is made to deprive the contractor of redress in case the conditions are found to be different.

Another condition that has crept into contracts is that the engineer is supposed to act in a judicial capacity, and rightfully so. Nevertheless, he attempts very often—perhaps the lawyers insist that he do so—to give himself arbitrary and dictatorial powers. The contractor must accept his decision and, by the wording of the contract, has no appeal. The engineer at best is paid by the other side; he owes his position to that side. Never in the history of the world have men been of high enough caliber not to be influenced by such a situation; and yet an attempt is made to deprive the contractor of the right of appeal. Of course, that has been upset by the Courts. No engineer or lawyer can deprive a man of the right to go to Court and ask for relief for any injury from which he even thinks he suffers.

Various technicalities, however, are put in the contract, to the effect that claims must be made within ten or fifteen days after a loss is incurred, and in case the contractor does not do it within the time stated, it is supposed to act as a waiver and he is deprived of the right to go to Court. That has usually been upset by the Court, but there is danger in it. It is often impossible for the contractor to make his claim within the time set. His loss frequently cannot be computed until the end of the contract, and he should be allowed to file claims at any time until the work is completed.

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* Pres., Spencer, White & Prentis, Inc., New York, N. Y.

The speaker cannot see the logic of Principle (13). The owner is to retain a greater and greater sum of money as the contract approaches completion, on the theory that the contractor is more likely to fail at that period. At the same time the paper contains an argument on the value of sureties and that seems inconsistent. The surety is largely expected to ensure that the job shall be completed, and yet it advocates the retention of a large part of the money for work actually done. A man should be paid for what he does, and no large amounts should be retained. He risks his capital and his reputation; he puts up a bond. Why should he not be paid for work actually performed?

A bank is apt to press a contractor at the end of the work for the payment of money loaned him. That, no doubt, is quite proper; but, at the same time, if the money he has earned is not paid him, how can he pay the bank?

The author discusses bid bonds (Principle (35)). To require a bid bond is in a way asking the surety company to do something with which it is not familiar. The surety company is required to guarantee a contract of which the price has not yet been made. The bid price, for instance, might be for $5 000 000 on a $15 000 000 contract, and yet the bond holds. That is really an injustice.

The number of contracts let is constantly increasing, and work is continuing in larger measure to be let by contract. Now, the reason it is not done directly is that it cannot be done so well, nor so economically as by contract. The engineer, whether he represents a community or a company, does not have the same freedom that a contractor has in selecting his men. He is hampered by various limitations and by the politics of his own community or the politics of the company he is with; whereas if a contractor operates on any such principle he is quickly eliminated by his financial losses.

The author describes an ideal contract. Those who examine the contracts recently let by the Board of Water Supply of New York City will find a fairly close approximation to the conditions given in this paper. No matter how perfectly a contract may be drawn and its conditions described there remains a very strong economic factor for the contractor to evaluate and that is the efficiency of his own force. What is the scale of wages going to be for the next few years? What will be the price of materials? These risks he is willing to take; that is his business.

A. E. Waldron,* M. Am. Soc. C. E.—This paper is very interesting, but the speaker fears that, if it is accepted, it will lead the engineer into a pitfall. It raises the question of the relation between the bonding company and the engineer. It sets forth many points to be covered by information to bidders, the specifications, or the contract. All these factors, no matter how desirable they may be, are those which the bonding company would like to have the contractor know. However, each item of information the engineer gives increases his responsibility to some extent. The speaker doubts that much information tends to reduce prices. When the contractor knows all hazards to be encountered, he is apt to bid higher than if he has to take a chance here and there. When chances are to be taken there is always a shaving of prices in order to "beat the other fellow". Moreover, when the engineer gives a great mass of information, much of it is likely to be speculative and may eventually prove to be incorrect. Then, if the contractor fails, the bonding company has good grounds to plead lack of responsibility because the information given was erroneous and the contractor did not find conditions as specified. The safe course is to give only such information as is absolute fact and is certain to reduce bid prices and not that which the bidders can ascertain for themselves through reasonable deduction.

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* Col., Corps of Engrs., U. S. A.; with N. Y. C. R. R., New York, N. Y.

With all the information given, as recommended by the author, the bonding company has nothing to do but to collect its fee and rest content. If the contractor goes wrong, then, the engineer is sure to have blundered in some of his information and the bonding company's responsibility is nil. It is not bonding the engineer.

A legal contract is defined as a meeting of minds. After such meeting of minds—that is, after a contract has been let—what is the usual procedure? The engineer tries to get as much as he can for his employer; the contractor tries to do as little as possible in order to increase his earnings. With all conceivable information given, thereby removing all deductions, the contractor becomes nothing more than a labor and construction boss. For satisfactory work under contract, the successful bidder should be more than this; he should bring some engineering ability to the job. If not, there are no good reasons why the engineer, knowing all conceivable conditions, should not do the work under his own direction or by hired labor, etc. All the things a contractor knows in reference to his job, the engineer should know to be able to decide whether or not bids are reasonable. Therefore, why should he not do the work himself?

In the case of public work there are conditions that make a contract desirable, but similar conditions make it, also, desirable from the bonding company's point of view, for the engineer to specify everything. On private work, however, the contractor's profit, the bonding company's profit, and the various contentions that arise, are all inducements for the owner to do the work himself.

It is stated that the engineer should keep the bonding company informed of various things and facts. The speaker cannot see the wisdom or profit in such procedure. What responsibility does the engineer have to the bonding company? It is only when the contract is annulled that the engineer should begin to deal with the company. To do otherwise, especially on public work in which money that is disbursed is passed upon by comptrollers, is "a step in the wrong direction".

At times, it seems as if lawyers have made engineers afraid to back up their work. However, the real situation is that engineers have learned that it is useless to buck the procedure that causes changes in the phraseology of specifications. The lawyer's word or advice is accepted and a superior officer gives orders that such and such be incorporated in the specifications, because some judge or jury decided a particular and restricted case in a certain way, and thus specifications include peculiar provisions. There are errors in law as well as in other professions. It is not the facts that lead engineers astray, but the deductions made from facts.

A. JORDAN BERNSTEIN,* M. AM. SOC. C. E.—The speaker considers this paper to be most excellent and one of the most interesting that he has ever read. He wishes to discuss it only as it applies to building construction in which the owners are individuals or corporations and not governmental agencies.

With reference to the method of determining the proper monthly payments to be made to a contractor the author states in Principle (12):

"A much better practice is to have the contractor, as soon as the contract is signed, submit a schedule of all the component parts of the work with prices attached, the total of which will equal the lump-sum contract price."

The speaker approves of this method, but he would caution the engineer against a practice on the part of some contractors who arbitrarily declare high prices for items that are done near the beginning of the job and low ones for those that occur near the end, for obvious reasons.

In Principle (13) the author makes the point that the owner needs to retain a greater reserve near the end than near the beginning of the work on account of the greater likelihood of a default at that stage. There is one condition near the very end of the job under which the suggested retention of a greater reserve works an unjustifiable hardship on the contractor. The work of putting the finishing touches to a building sometimes stretches over a considerable length of time. This may be due to a lack of energy on the part of the general contractor; but more often it is caused by the delay on the part of a contractor who is employed by the owner directly and over whom the general contractor has no control. At other times, it may be due to prospective tenants whose decisions with regard to partitions, store fronts, etc., must be taken into account. Whatever the cause, the general contractor is faced with the unpleasant fact that, although the items that still remain to be done may represent only a few thousand dollars, he may have a huge sum tied up in the 10% or 15% reserve, on the technical ground that he has not finished 100% of the work.

This safeguard of retaining a reserve, as well as most of the safeguards to be found in the average contract form, is in favor of the owner as against the contractor. Apparently, little thought has been given to protecting the contractor against an owner who has over-extended himself financially and who compels the contractor to accept paper as payment after "stringing him along" for a considerable time subsequent to the completion of the job.

In Principles (17) and (25), the author urges the engineer to furnish drawings and specifications that are more detailed and complete than is often the practice, and to furnish more information regarding local conditions. While the speaker agrees with this recommendation, he is of the opinion that there is little cause for complaint on this score when the engineer is allowed enough time for the preparation of his drawings and specifications, and more especially when he is paid adequately for his services. Unfortunately, the engineer often accepts work at too low a price and frequently agrees to complete his work in a very short time. Under those conditions the plans cannot be entirely complete and the engineer is generally compelled to copy the specifications of a previous job, making only such modifications as suggest themselves in the hurry of getting the drawings and papers ready for bids. Discrepancies between plans and specifications are the result.

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* Cons. Engr., New York, N. Y.

The author makes an excellent point when he advises the exclusion from contract forms of clauses which properly belong under "Information for Bidders" (Principle (22)). It is surprising how persistently contract forms which are replete with irrelevant matter and so poorly worded that they cause inconvenience to all parties, continue in general use.

In Princple (37) the author urges the public opening of bids even on private work. The speaker approves of this method, provided only that it is recognized by all parties that there may be a great difference in the capacity, the willingness to co-operate, the sense of fairness, and the character of the several bidders (all of whom may be entirely responsible), and that it is proper, therefore, to pay to one contractor somewhat more for a particular job than to another.

The author recommends that the contract be awarded to the lowest bidder if he is qualified, irrespective of what a local contractor has bid (Principle (38)). The speaker would point out that the amount of a bid is not the only factor that determines the final cost of a building. An out-of-town contractor is more likely to get into difficulties with the local labor unions or with the municipal bureaus than a local man, thus possibly delaying the completion of the building and, therefore, increasing the owner's carrying charges. Furthermore, a local contractor can generally secure the best local mechanics. The out-of-town contractor, therefore, should expect to lose the contract unless his bid is considerably less than the local bid.

After erecting about sixty buildings in a dozen cities the speaker has come to the conclusion that the best written contract affords but little protection against a contractor who is incompetent or who is too preoccupied with other work to live up to a specified schedule. Therefore, he would recommend that the engineer draw the contract with the greatest care and provide every possible safeguard; but, above all, that he award a contract only to bidders who have earned a reputation for ability and for sterling character.

ERNEST NICHOLSON,* ASSOC. M. AM. SOC. C. E.—In reference to Principle (3), the speaker knows of several instances in which details were omitted from the plans so that they were incomplete when the bids were taken and the contracts signed; when finally completed by the architect they had been so elaborated that Court action was necessary to straighten out matters. Frequently, only simple details are required, but if these are lacking, it becomes necessary to bid high for protection to the contractor or sub-contractor.

In New York and Pennsylvania the charges for bond premiums vary as mentioned under Principle (7), in connection with public work, such as public buildings. The premium in New York is two-thirds that in Pennsylvania, the difference being due to labor and material liability under the bond in the two States.

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In reference to Principle (24), one State in New England will not allow a contractor to bid on several road jobs at the same bid opening with the option of withdrawing from some of them. In New York State bidders are allowed to withdraw other tenders if they are low on one or more—the withdrawing being necessary as soon as the bids on the previous job are read. In Pennsylvania, the extensions and additions are checked by the State engineers before the following job is read, and the contractor is then sure of his arithmetical work before being called on to decide if he wants his other bids to be considered. It is obvious that in New York and Pennsylvania a small operator has a better chance of securing a job and the States, in turn, have the advantage of securing lower bids at times,

An example might be quoted in support of Principle (26). In connection with a certain State road contract, the job was held up by an injunction just after the contract was signed and the bond delivered. After considerable delay, and when it was thought the work would never be built, the contractor took on another contract in an adjoining State. Later, the objection to the first contract was removed and the contractor tried to run both jobs, but this volume of work was too much for him and he defaulted.

Regarding Principle (27), it is known that when estimates are given out prior to the bid opening, some contractors of doubtful experience, ability, and resources put in bids on the basis of a certain percentage of the engineer's estimate.

In reference to the furnishing of quantity surveys, as mentioned in Principle (28), the speaker knows of one city where the contractors bidding on a job divide the cost of having a survey of quantities made for the use of all, in order that such cost would not be levied on each. Many small contractors compute two and three jobs per week, the cost of which must finally come out of a future job. In some places the contractors sign an agreement to add a fixed sum to their bid and then the low bidder shares this amount with the unsuccessful ones. The speaker understands that in England it is becoming the general practice among architects to furnish bidders with a detailed quantity survey.

ARTHUR W. CONSOER,* M. AM. SOC. C. E. (by letter).—The author presents an argument in favor of simplifying and rationalizing the letting of contracts by competitive bidding by improving the quality and completeness of the plans and specifications and contract forms prepared by the engineer. This is intimately connected with the problem of adequate engineering fees for practicing and consulting engineers and adequate appropriations for the work of public or corporation offices engaged in engineering work. Improvement in the scale of compensation paid to practicing and consulting engineers and to engineering staffs will automatically result in marked improvement in the preparation of plans and specifications and contract forms. Such improvement, in turn, will result in savings in construction costs and reduction of litigation with surety companies and other parties.

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* Vice-Pres., and Gen. Mgr., Consoer, Older & Quinlan, Inc., Chicago, Ill.

Rigid inspection and able engineering supervision are costly. An engineer who is operating on a low fee basis in a highly competitive field is very apt to specify results which the contractor must produce and to demand unreasonable maintenance guaranties instead of merely specifying workmanship and materials and relying on his own rigid inspection and supervision for satisfactory results. Inexperienced engineers, likewise, hesitate to assume responsibilities for their own designs and lean on the contractor by requiring him to be responsible for the final results. When unsatisfactory results on construction work are due to poor engineering design, the contractor should not be held liable under the contract.

Frequently engineers do not have sufficient control over the inspection staff. On public work, particularly, the appointment of political favorites as inspectors is regrettably common. Under such conditions engineers sometimes hesitate to assume full responsibility for the sufficiency of the plans and specifications, and attempt to solve the difficulty by exacting maintenance guaranties from the contractor and his surety. The practice of strengthening the position of the engineers in charge of public works by giving them closer control of the inspection staffs will facilitate desirable changes in contract forms in public work.

The cost to contractors of preparing proposals for competitive bidding can be reduced by improving the diction and "make-up" of advertisements for bidders. The author's suggested form of advertisement is clear and concise. However, the writer would suggest further that it should include a statement as to length of time that will be allowed for the construction period and that the manner of payment, whether in cash or bonds, should also be stated. On large projects it is helpful to the bidders if, with the plans, specifications, and proposal forms, there are included some airplane photographs and other views of the site of the work. There should also be a detailed sketch showing available railroad sidings for unloading materials, with information regarding rental charges for such facilities, and locations of approved available supplies of sand, gravel, or other materials, as well as a detailed list of approved material sources within convenient freight haul of the site of the proposed work. When underground or foundation work is involved, information should be supplied by the engineer in the form of descriptions of test pits or soil borings. If quantities given in a proposal form are known by the engineer to be in excess of the amount that will go into the work, prospective bidders should be apprised of that fact. Competition will be increased and prices lowered if the engineer will eliminate all lump-sum proposals and take bids only on the basis of detailed, accurate, quantity surveys. Such detailed lists of quantities are not only helpful in reducing the contractors' cost in preparing proposals, but also to the engineer in his preparation of estimates for payment to the contractors during the construction period, and in checking the accuracy of final estimates for payment based on actual measurements in the field.

A large source of expense to many contractors is the tendency of many engineers to inspect materials of construction on the site of the work only and to forego plant inspection. If specifications were written to provide for plant inspection the contractors on many projects would be able to reduce their estimated costs. The practice of culling paving brick and sewer pipe on the street has made many a contractor insolvent. Specifications for the qualifications of inspectors and field engineers might also be included with advantage. If a contractor bidding on a project knew that no one would be employed to inspect his work without having had at least five years' experience on construction work of a similar nature, he might be able further to reduce his estimated costs.

Whenever a new contractor bids on the writer's work, he is supplied with a copy of the "Standard Instructions to Inspectors" used in the writer's practice, so that such a contractor may gain some idea of just what type of inspection he may expect to encounter on the work. Many a foreign contractor is kept off a large project because of his lack of knowledge of the methods to be used by, and the characteristic attitude of, the engineer in charge. Anything that can be done by an engineer to remove all uncertainties regarding himself and his subordinates will increase competition and reduce construction costs.

Engineers, contractors, and surety companies are all interested in reducing waste in the construction industry. If the engineer can effect savings for a number of contractors by supplying certain information, rather than by forcing all bidders to procure such data individually, he should do so. Likewise, many of the reforms in administration of contract work let by competitive bidding, which are proposed by the author, will eliminate waste and reduce construction costs.

Most of such improved practice in letting work by competitive bidding will reduce construction costs and at the same time will increase engineering costs. The aim should be to strike an economical balance between the two factors. In many cases it will be found possible and economical to double the amount spent for engineering work without increasing the total cost of the project, inasmuch as engineering costs are a small fraction of the total.

F. H. Stephenson,* M. Am. Soc. C. E. (by letter).—The author has presented some very pertinent recommendations regarding a subject of major importance in engineering practice. Complete discussion of this paper will probably reflect the viewpoints of the engineers of the bonding companies, the contractors, and the owners. As an employee of a municipal corporation, the writer has the following comments to offer.

The author recommends that in the "Advertisement for Bidders" an approximate estimate of the cost of the work be published, giving an upper and a lower limit. The writer fully believes that, by reading the advertisement, the prospective bidder should be able to decide whether the work on which bids are requested is of the kind in which he is skilled and experienced and of a size to correspond with his financial status.

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* Engr., Water System, Dept. of Water Supply, Detroit, Mich.

On the other hand when State or municipal work is involved contractors cannot be refused plans and specifications. Very often these inexperienced people are anxious to get the engineer's estimate as a check on their own calculations, which may sometimes be wild guesses rather than figures backed by experience and former cost records.

With all things considered, the writer believes that a rather wide variation between the published price limits would often be a help and, at other times, a hindrance. In view of the wide divergence of bids made by contractors of apparently equal experience and financial standing, the upper price might well be 50% greater than the lower price.

The writer agrees with the author in Principle (34) that a designated lump sum as bidding security is preferable to a fixed percentage of the bidder's proposal to do the work. He believes, however, that the relation of bidding security to total cost should be a variable amount, depending on the size of the job. The sum of 10% on a $1 000 000 job is considerable to hold as a certified check for two weeks or more while perhaps the responsibility and ability of bidders are being investigated. It seems reasonable and fair to stipulate that if the bidder fails to execute after being awarded the contract he should be penalized only the actual loss to the owner by accepting the next highest bid.

The writer's experience does not accord with the author's statement that "labor really does not need protection because it will not work unless payrolls are promptly met, * * *." Labor really needs even more protection than it is given at present. Material men also need protection, and the writer's experience has led him to conclude that even a bond does not furnish all the protection it should. The owner—whether it be an individual, corporation, utility, or unit of government—is primarily interested in getting certain work done at a fair price, and he desires to assist the contractor by helping to secure labor and material and at the same time to protect that labor and material from loss through no fault of either party.

JOHN C. WAIT,* M. AM. SOC. C. E.—The author presents two conditions for the successful completion of contract work: (1) The choice of a capable and experienced contractor who is thoroughly reliable, financially and otherwise; and (2) the choice of a good surety company that is thoroughly able and willing to back him. Every one will agree that this would be an ideal condition for the owners, municipalities, States, and the contractors, but, practically, it does not exist.

Especially, in the City of New York, some of the greatest projects and the biggest contracts have been undertaken by unknown contractors without personal finances. Some contractors have done New York's greatest work and have gone, penniless, to the grave. Some of the greatest buildings, bridges, tunnels, and subways, have been successfully built by men who could scarcely be expected to undertake great work.

The policy that is recommended in this paper (see Principle (39)) is now developing in the municipalities and in the States; that is, the policy of investigating the financial strength and the experience of contractors.

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Public officials are being authorized by law to determine the reliability, responsibility, and capability of contractors, and they are doing so to the fullest extent. The great Manhattan Bridge was built by a contractor who owed his bank thousands of dollars; who did not even have funds with which to begin the work; who was opposed by all the steel companies in North America; and yet who built the bridge in the shortest time in which any suspension bridge had ever been built, and there was not one single complaint or lawsuit that followed its completion.

Now, why select contractors who are successful, who have great financial and political backing, and who have great experience, when there are others who are not so fortunate nor so successful, but who can carry out the contract. Some of the biggest public works being built to-day are by contractors who were not known to have financial backing or experience, and the contracts are being successfully completed. Low prices do not always come from big and successful contractors.

The speaker is in sympathy with a great many suggestions in this paper, but he believes that they apply largely to private work. Some of the recommendations made cannot be accomplished, especially on public works. For instance, contrary to Principle (5), the engineer cannot relieve surety companies on public works from giving a bond to pay for the labor and materials expended, because the statute laws of the United States and of some States require that such a bond must be given.

Principle (4) is a sound one. According to law, an owner may not specify the manner and method of doing the work and, at the same time, stipulate the materials to be used and then exact a warranty of results from the contractor.. It is dangerous and productive of litigation to specify both how and of what materials the work shall be done and then say that the contractor "guarantees" or warrants it. The Court says that, if the structure leaks, although the contractor "warrants" it, he may not be held to the warranty. The speaker distinguishes between the words "guaranty"—which is a promise to pay the debt of another man—and "warranty"—which is a warrant of the work done and materials used.

The engineer usually prepares the "Information for Bidders". He makes soundings or borings; he determines quantities; the elevation of water surfaces or sub-strata; the character of excavations; the presence or absence of materials of construction, etc.; at great expense to the owner or municipality. Then what is done?

The contractor is told that nothing is definitely known about these quantities and conditions; that the owner will not be held liable; and that the contractor must be responsible for them. In a week or ten days he must get information that the engineers may have taken nine months to collect at a cost of thousands of dollars. That is a universal practice in advertising for bids, on either public or private work.

It is done to protect the owner from the possibility of adverse Court decisions if the judgment and knowledge of the engineer should prove to be inexact. As the author states, "nobody can see very far under water, or into the earth"; but the engineer can judge better than the contractor. Owners may be sure, therefore, that the contractor will add to his price because of the repudiation of the engineer's estimate. (See Principle (3).)

The speaker suggests writing to 100 or 1 000 contractors and asking how much they would decrease their bids to a municipality if they had a positive declaration as to quantities and conditions existing on the proposed work and if they were not required to guess or take information from engineers only to have it later repudiated in the "Notice to Bidders".

To deny, or refuse, to support the engineer's report, is an act that deprives the owner or municipality of the money spent for engineering and investigation. The contractor must then be responsible for engineering data obtained by the owner, and he adds a liberal amount to his bid or contract price accordingly. In municipalities the practice has been promulgated by the legal departments in an effort to forestall or prevent successful litigation by the contractor.

The speaker has sometimes believed that the engineer quite approved the practice because it shifted the responsibility for errors in engineering work. The practice is justified by attorneys because when a contractor has obtained a large judgment against the owner, through engineering errors or misjudgment, it is a definite lump sum that is apparent and is felt by the defeated attorney; whereas the untold sums added by contractors to their bids are not so apparent. They are, nevertheless, met in even larger quantities by the owner or municipality.

The trouble with engineers is that they allow the lawyers to dictate to them what shall be included in the contract. Of what value is a report of investigations to contractors when the owner's lawyer (who prepares the contract) states that, in all events, the contractor shall be held responsible for the accuracy of the work of some one else?

The speaker believes that engineers do not realize how important the surety is to companies in the letting of contracts. The present tendency of corporations and municipalities is to make of the contractor a casualty insurance company by forcing him to indemnify the owner, not only for any negligence of himself or anybody that may act for him, but for anything at all that may happen—even under the direction of the engineer.

That is current practice in the City of New York. Where is there any justification for it? Again and again, the speaker has tried to have the legality of these clauses passed upon by the Court. State laws forbid anybody from writing insurance—casualty or otherwise—unless they are duly and properly authorized by the State and unless they accumulate and hold large bonuses. A contractor is not authorized to write insurance for the public or for municipalities, but the ordinary construction contract requires him to do just that very thing.

When confronted with this issue, judges of this State have avoided it by deciding the case on other grounds. Some day it will be decided definitely and, the speaker believes, that will stop the practice of requiring the contractor to act as an insurance company.

Edward W. Stearns,* M. Am. Soc. C. E. (by letter).—There is so much that is good in this paper that it deserves careful reading by those whose duties include the preparation of plans and specifications for contract work. There are, however, a number of points included which, in the writer's opinion, are quite controversial and there is, furthermore, one very broad principle which is quite axiomatic and which deserves greater emphasis and amplification than was given it by Mr. Bush. The method of treating the subject adopted by the author is quite ideal and lends itself admirably to reading, discussion, and reference.

Principle (1).—Fundamentally, Principle (1) is correct, but its application should be followed with certain discretion. In the preparation of contracts and specifications it frequently happens that a broad general requirement, clearly and fully stated in its proper place, should be emphasized later by mention in special items subject to the requirement. When this is done, of course, it is of the utmost importance that each subsequent statement does not disagree in the slightest manner from the broad requirement first stated. No doubt a number of writers, and more particularly those of the legal profession, will take exception to this practice on the theory that by mention or emphasis of any particular application, the points not mentioned are, by inference, weakened if not entirely invalidated. In order to overcome this objection it is the writer's practice to include a clause of the following nature in the contract:

> "The enumeration of particular instances in which the opinion, judgment, discretion, or determination of the Engineer shall control or in which work shall be performed to his satisfaction or to his inspection or of particular items to be performed without additional compensation, shall not imply that only the matters of a nature similar to those enumerated shall be so governed and performed, but without exception all work shall be so governed and so performed."

In the case of contract documents for a building, the broad requirement is laid down that the contractor shall comply with all Federal, State, and municipal laws and regulations and shall secure all necessary permits. These documents are prepared with the intention in mind that there will be a single general contractor on the work and that it is his obligation to comply with this provision. There will, however, be many sub-contractors engaged in the construction of the building, and in all likelihood the general contractor will expect each sub-contractor to comply with the laws prescribed and to secure such permits as are particularly required for his trade. It seems entirely proper, therefore, to indicate in the sections of the specifications pertaining to each trade, that the owner expects the contractor, meaning the general contractor, to see to it that all laws and regulations are complied with and that all necessary permits are secured, so that the sub-contractor shall be aware of the requirement and that he may determine his obligations from the general contractor.

Principle (2).—The writer desires to add to the discussion regarding Principle (2) a consideration of "cost plus" work. Many people engaged in construction work seem to have little conception of the fact that a very important item in a contractor's cost is the question of overhead and that there is no fixed method among all contractors for the apportionment of overhead expenses. When the contractor is called upon to do work on the basis of cost plus percentage a definite agreement should be reached before any work is done, as to just how properly to compensate him for his overhead expenses and just how the item of cost will be measured.

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It should be borne in mind that usually "extra work" is not undertaken because of the desire of the contractor for it, but because the owner finds it the most convenient way of handling certain minor matters. With this thought in mind it is only equitable that the contractor should receive not only his actual direct outlay in cash for labor and materials, but also both his overhead expense and a fair profit. By clearly understanding at the start just how the word, "cost", is to be defined, much controversy and ill feeling can be avoided and, as a result thereof, better work can be obtained due to a more sympathetic feeling between the contractor and the owner.

Principle (6).—Probably the greatest uncertainties which a contractor assumes in any construction work are those which are incident to foundation conditions. Those are the uncertainties which cannot be seen and must be felt. The writer thoroughly believes in sub-surface explorations, commensurate with the importance of the work involved, and believes further in furnishing to the bidder the complete information which he obtains from such explorations exactly as he receives it. It is obvious that his plans represent his interpretation of the results of such explorations. The writer does not believe in guaranteeing to the contractor either the correctness of the sub-surface explorations or that the conditions indicated by them will be found actually to exist; or, in other words, that the interpretation as indicated by the plans is correct. Those risks must and should be taken by the contractor.

As an example, the writer recalls a job on which extensive borings indicated a thick stratum of soft clay overlying a bed of gravel or hardpan. The plans required that the piers be supported on piles. After studying the samples from the borings, the contractor ordered piles of sufficient length to be driven through the clay and into the harder strata below, but in driving it was found that the piles would penetrate only partly into the clay. The cost of the pile cut-off was placed upon the contractor, the contract being a unit price job with a price per foot for piles driven. This ruling may seem to have been a hardship on the contractor. It should be borne in mind, however, that the full information furnished from the borings was in his possession at the time he prepared his bid, and, further, that he did not elect, before ordering his piles, to drive a few test piles by which he could have determined the probable penetration. Consequently, he assumed the full risk when he ordered the long piles and should not have been compensated for the loss incurred in the pile cut-offs.

The foregoing, however, does not, and in Court cannot be made to, hold if the sub-surface explorations are obviously insufficient or grossly incorrect. The writer recalls one job on which borings indicated rock at such an elevation below water level that the excavation could presumably be done by the open coffer-dam method, and the contractor so calculated his bid. When the so-called rock was exposed, however, it developed that it was not bed-rock, but a rather unusual layer of large boulders. Subsequent exploration then revealed the fact that bed-rock lay at a depth so much greater than that orginally contemplated that pneumatic caisson work became necessary. It is obvious that, to force the contractor to perform the pneumatic work at the price bid for coffer-dam work would have been inequitable, and could not have held in Court.

The foregoing should not apply to sub-surface man-made structures. The engineer should endeavor to determine to his best ability the existence, location, and character of all such structures and furnish such information to the contractor, thereby making him solely responsible. Should structures not shown on the plans be encountered, the risk thereof and any work involved in properly caring for them should be solely at the cost of the owner. As an example, on one job in the writer's experience there was encountered a 24-in., high-pressure water main which was not shown on the plans and which had to be moved. One entire section of the city was dependent almost wholly on this main for domestic use and fire protection. The removal of the main from the vicinity of the work was not an expensive operation, but the risk entailed was tremendous if the main had been broken or if a fire had occurred at the time necessary to shut off the water. Obviously, here the equity lay only in the owner assuming both the cost and the risk.

Principle (7).—The surety premium represents the charge for carrying certain risks. Obviously, then, if the risks are reduced the premium will be reduced and Principle (7) is only a statement of this fact in regard to particular risks. The writer believes, however, that the State lien laws are based on sound thought and, therefore, that the State and Government should provide the same protection as is given to the laborer or material man on private work. The risk for the payment of such claims is clearly one to be covered by insurance and, therefore, the surety should be required to guarantee the cost of unpaid labor and material bills.

Principle (*13*).—Except when there is no surety bond—and it is the writer's opinion that every contract should include a surety bond—the "retained percentage" is not withheld by the owner as a safeguard in case of default. On the contrary it is withheld for other purposes, as follows: (1) To cover incidental work of minor importance usually deferred until the end of the contract; (2) to serve as a liquid fund in case the contractor defaults; and (3) to act as a counter fund to offset any possible over-payment due to the approximate nature of the monthly estimates. Other reasons of lesser importance might be mentioned.

The writer believes that in the early stages of the contract the percentage which the "retained percentage" bears to the cost of work done, should be relatively high in order to build up a considerable cash fund, but that, as the work progresses and nears completion, both this percentage and the amount of the "retained percentage" should decrease until, when the work is entirely complete, they both should be relatively small. In one contract, the writer specified that 10% of the value of the work performed be withheld until such a time as the retained amount equaled 5% of the total estimated contract price, after which time no further amount should be withheld. In another large contract he specified that until 25% of the work had been done, 10% should be withheld, after which the percentage was reduced on a sliding scale until, at the completion of the work, only 2½% was retained.

There is a feeling among many engineers and architects that a contract should be worded so as to force the contractor to finance himself very extensively. To this opinion the writer takes strong exception. He believes that monthly payments should be arranged so as to compensate the contractor as far as possible for the cost of the work as it proceeds. Usually this cannot be done in a simple way. In the early part of the work the contractor necessarily has a heavy investment and a relatively small amount of work performed so that his estimates do not and cannot fully compensate him. It is for this reason that his financial ability is important. After the work has progressed, however, if payments are properly arranged and if the retained percentage is not exorbitant, the time should quickly come when the payments should carry the contractor's cost.

Usually, the owner is in a much better position to secure the use of funds than the contractor. By compensating the contractor, fairly and reasonably, as the work progresses and reducing his necessity of obtaining loans, sometimes at quite high rates of interest, the owner establishes that feeling of good-will between himself and the contractor which is conducive not only to greater effort on the part of the latter to get his work done well and expeditiously, but also to cause him to reflect such feeling in his bid price. This is particularly true when the owner is in the market at more or less frequent intervals for construction work; but it is nevertheless also true when the owner is contemplating and planning only one piece of construction work, providing the contract and specifications are so worded as to show clearly to the bidder that he can expect fair treatment.

Principle (17).—It is with considerable humiliation as an engineer that the writer discusses this obvious principle. Unfortunately, the competition in engineering work has become so acute that the fees for it have become proportionately meager. Commercial organizations frequently seem to be so short-sighted as to consider the cost of the preparation of their drawings and contract documents as a whole as unproductive costs. This is fundamentally wrong. Care, thought, and study cost money, but when they are applied so as to produce drawings and contract documents that reduce to a minimum the points in which the contractor must use his imagination, the money so invested is saved many times over.

The engineer or the architect, whether he be employed as an outside agency or whether he be a regular employee of the owner on salary, has imposed upon him a great trust, namely, to construct for the owner, at the lowest possible cost, a structure which can best serve the owner's needs. Such results can only be obtained by the careful and thorough preparation of the drawings and contract documents. When he lends himself to any other practice on the grounds that he does not receive sufficient funds to justify such careful and thorough preparation, the engineer or architect is breaking his trust most disgracefully and lowering the standard of the profession as a whole.

Principle (21).—The writer believes that it is best not to state the estimated cost of the work, even with limits as suggested by the author. The particular limits to be adopted would be subject to considerable variation in practice at different offices, and, if made broad enough to provide the possible range of price as found in bids for the majority of work, they would be of little value in indicating its size. A general statement of the approximate nature and dimensions of the work contemplated would seem to be a better basis for indicating its magnitude and character to the bidder.

Principles (34), (35), and (36).—It is essential for the writer to consider these three principles together. He takes issue with the author on four points, namely: (1) That the cash deposit required of the bidder is primarily required as a "bidding security"; (2) that it should be as much as 10 or 15% of the "probable contract price"; (3) that bid bonds are not desirable; and (4) that bid letters are not desirable.

The writer believes that the best bidding security is the bid bond or the agreement of proposed surety which is called a bid letter by the author. Since the responsibility of the surety, who must be acceptable to the owner, is apt to be greater than the responsibility of the bidder himself, in the writer's opinion the cash deposit at the time of submitting the bid has the chief purpose of giving a measure of the contractor's financial ability. Consequently, the amount of the bid deposit should be only sufficient to indicate that the bidder, if awarded the contract, has sufficient liquid capital or credit to proceed with his contract without embarrassment to the owner. Before awarding a contract the owner should require the contractor to submit an up-to-date financial statement. This, together with his bid deposit, should furnish the owner with sufficient information of the bidder's financial resources. It is difficult, if not impossible, to fix a formula for determining the amount of the bid deposit. It has been the writer's practice to fix it somewhere between 3 and 8% of the estimated cost of the work, depending on the class of work, the cost, and the length of time to be involved in completing it.

In conclusion, there is one broad principle applicable to the preparation of plans and contract documents which is axiomatic, but which nevertheless seems only too often to be entirely overlooked. Mr. Bush touched on it a number of times, but failed to emphasize it by setting it among his other principles. Briefly it may be stated as follows:

"Clarity and consistency are of prime importance".

Close attention should be paid to this principle, particularly in the preparation of the contract documents. Although, due to tradition and practice, certain legal phrases more or less difficult for the layman to read seem, by custom, to be unavoidable, the preparation of such documents nevertheless is only the practicable application of the English language. It is obvious of course that the individual preparing the contract documents must know what he wants and, to a certain extent, how to get it. Unfortunately, however, the documents are often so worded as to leave the contractor completely ignorant of the real intent. The point cannot be too strongly emphasized that every statement in such documents should be carefully weighed with a view to determining whether or not they correctly carry the thought which the writer desires to transmit. As a rule, the more brief and concise the documents are made, the more intelligible they are to those using them; but this consideration should not be carried to such an extreme that much is left to inference or interpretation.

Only too often, the writer fears, is it true that contract documents previously prepared—possibly even with careful analysis and study—are used as a guide in the preparation of subsequent documents. This practice often results in inconsistencies which are sometimes serious and many times quite ludicrous. The writer has had to review a specification prepared by another engineer, in which an entire chapter was devoted to a description of a type of work with which he was unfamiliar. Throughout the specifications reference was made to this work, thereby giving the impression that it was of considerable importance. At the first opportunity he questioned the man who prepared the specifications as to the details involved in this particular type of work, and after getting a reasonably clear conception of what was intended he inquired where it was to be used in the particular construction contemplated. Much to his surprise he discovered that none of it was contemplated, and the entire chapter and the references to it had merely been copied from a specification for an entirely different piece of work. Such practice is in much the same category as that of the calculator who claimed that he made only a very minor error in his calculation when he misplaced the decimal point. To some, the foregoing criticism may seem ridiculously obvious, but when one has encountered such conditions not occasionally but frequently over a period of years it becomes a very serious matter.

The preparation of drawings and contracts is only part of the story. After having attempted to emphasize the importance of fairness, clarity, and consistency in the wording and treatment of such documents a word to those whose duty it is to execute them and carry out their provisions is not remiss. In all the work that the writer has ever done he has made it an unvarying practice to convey to the contractor in one way or another the feeling that the contractor and the engineer are both working to the same end, with the same purposes in mind, and with the same obligations, namely, to complete the construction, whatever it may be, in the best possible manner, in the shortest space of time, and at the least possible expense both to the contractor and the owner with the intent of the plans and specifications and the purposes for which the structure is to be used. All the contract documents, therefore, become only a portrayal of the broad general principles under which the work is to be performed. It behooves the engineer in charge to establish with the contractor, by fair dealing, fair interpretation of the contract provision, and sympathetic consideration of the contractor's problems, that spirit of close co-operation and good will which alone will result to the best interests of the owner.

Too many engineers, from the lowliest inspector to the chief in charge of the work, seem to have the impression that the provisions on the contract documents are immutable; that they must be followed to the very letter rather than to their intent; and that their prime obligation in carrying out the work is, if not completely to break the contractor, at least to see to it that he makes no profit. The contractor is entitled to a profit; to all the profit he can make, consistent with thorough and first-class workmanship. To deny him that profit by means of unfairly rigid and restrictive rules and by narrow-minded and short-sighted interpretations is the best way to create an unhappy condition in the execution of the work, a feeling of deep distrust and, in cases, even dishonesty on the part of the contractor, and, finally, expensive and uneconomical work.

Richard S. Kirby,* M. Am. Soc. C. E. (by letter).—The writer is in thorough accord with almost the entire paper. He has been struggling for years to inculcate just such principles into successive generations of engineering students. In his own mind he has classified Mr. Bush's "thirty-nine articles" into five groups, as follows:

(*a*).—More orderly arrangement of the material forming the contract documents: The writer suspects that much of the confusion to which the author properly calls attention has resulted either from leaning too confidently on precedent or from a division of responsibility in the preparation of the documents. Neither condition is typical of the engineer at his best.

(*b*).—A great degree of explicitness in the wording of the documents and in the data shown on the drawings, so as to remove uncertainties: To the writer's mind this group needs all possible emphasis, and Principle (3) is one of the most important of the entire thirty-nine, and the most difficult to put into practice. The writer would phrase Principle (9) differently—"Prepare each contract so carefully that nothing will remain for the Court to interpret." Courts and lawyers flourish on agreements entered into loosely. Construction work contracts ought to be so complete that if the contractor, owner, and engineer should die the day after a contract is signed, their successors would nevertheless find themselves in a position to carry out an agreement which did not lack precision at any point. The writer agrees especially with the suggestion that the graphic language be used in greater variety on plans. (See Principle (17).) Isometric drawings are so easily made and, at times, are so effective that it seems a shame that they are not used more often to supplement orthographic views.

(*c*).—More thorough and extensive engineering investigation of the project in advance: The writer agrees heartily with this general principle. He differs with Mr. Bush on Principle (27) only, as he can see no serious objection to percentage-unit bidding, provided the engineer gives proper thought to the preparation of his estimate.

(*d*).—Uniform treatment of bidders; improvements or changes in letting procedure: Doubtless the author had private, rather than public, work in mind in some of his suggestions; and it may be that architects are greater offenders than engineers in these respects. The writer questions the practicability of Principle (24) only.

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(*e*).—Reduction of sureties' duties, responsibilities, or liabilities: Some of this group of suggestions raises doubts in the writer's mind, especially Principle (39). It is plainly within the engineer's province to study carefully any matters concerning physical conditions relating to a project and to inquire into a contractor's reputation for good work, judged by past performances. How far an engineer should pose as a financial expert is another matter. Is not this plainly within the scope of the service which a surety company is expected to render?

J. E. ROOT,* M. AM. SOC. C. E. (by letter).—This paper contains many points of active interest on the letting of contract work, and the relations between engineers, contractors, and surety companies.

Preparation of Contract.—The ideas presented in Section *A* cannot be emphasized too strongly. They practically cover the full field of controversy between the contractor and the owner or his representative, the engineer. The contract form should be brief, clear, and applicable to the work at hand; and, as an instrument, it should represent mutual regard for justice and fairness for each party to the contract. In construction, the instrument should be arranged in proper sequence, of general application, and should contain only matter relative to the particular case at hand. In brief, the one principle for the engineer to follow during its preparation, in order to fulfill completely the objective and purpose which the contract is to serve, should be to make it a logical, comprehensive, readable, reasonable, and workable form.

In the construction of a complicated piece of work the question of "changes" does arise. However, the contract form and the detail specifications need not, and should not, emphasize glaringly that the engineer reserves full authority to use the prerogative of continually harassing the contractor by making constant changes and modifications during the period of construction. The engineer's full responsibility exists during the time in which he is preparing the contract form, specifications, and plans. If, during this interim, he will give the same thought, care, and attention to the contract form, specifications, and plans, which he gives and exercises, in too many examples, after the award of the contract, a better workable understanding and a more harmonious working arrangement will exist between engineer and contractor. If the contractor is ordered by an engineer, or one of his subordinates, to make changes from day to day as the result of plans inadequately prepared, or to modify specifications not prepared to fit the work at hand, how is it possible for a contractor to organize his forces, follow a construction program, use his initiative and ingenuity in the execution of the work, or obtain in the end that to which contractors are entitled, a reasonable profit? The time for changes in the contract form, specifications, and plans is prior to advertising for bids, and the principle upon which engineers should work should be that no changes are to be made after the contract has been let. If they would adhere to this principle it would immediately establish a definite line of demarcation between the engineer's functions and the contractor's functions during the life of the contract.

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Small scale drawings bound with the contract document serve no useful purpose. If it is necessary for the engineer to depict the work to himself by large scale drawings and to correlate all features and details of the problem by various suitable scales, it cannot be presumed that small scale drawings will give full information and enlightenment to a contractor or his engineer representative. The legal connection between the drawings and the contract, can be established by direct written reference made in the contract or in the specifications.

The defaults by contractors are usually from one or a combination of the following causes: (1) Lack of experience or financial responsibility; (2) bid prices that do not represent the actual cost, plus a reasonable percentage for its execution; (3) continued interruption and interference on the part of the engineer; and (4) contingencies arising during the work, which are beyond reasonable expectations.

To meet the first and second anticipated possibilities it is encumbent on contractors themselves to use judgment and discretion in bidding on proposed work. Contractors as well as engineers obtain experience gradually by their increased initiative and desire to expand a little at a time from a moderate to an involved class of work. In time, contractors as well as engineers, establish for themselves a classification and should only be encouraged to compete for work of a class in which they have clearly demonstrated that their past ability will fairly represent the probable successful execution of the work at hand. On this basis it is immaterial whether a 10% or a 5% retainer is withheld for the owner's protection against possible default, because, when expressed in a percentage of the contract, it represents in part the item of profit to the contractor. Carefully prepared balanced bids usually provide adequate funds to meet current obligations and a continuation of the necessary banking credit. The third and fourth possibilities speak sufficiently for themselves.

Established custom may easily and conveniently be modified so as to provide protection for the contractor against the possibility of changes in plans made by an unscrupulous engineer. For example, all bidders should be allowed to retain the plans until ten days after the award and the successful contractor should be permitted to keep, as his personal property, the plans used to prepare his bid. All deposits made for securing the plans could then be returned, or disposed of as the instructions to bidders may provide.

Usually, the period between the advertisement for bids and the date upon which they are to be submitted is too short to give a reliable contractor sufficient time for a just and careful consideration of the work involved. After the completion of plans and specifications that have taken months and sometimes years to prepare, engineers are, as a rule, entirely too anxious to receive bids and get a contractor started on the execution of the work. This time period should be reasonable and should vary with the class of work on which bids are to be obtained.

Section B.—Under the caption, "Advertisement for Bidders", there is one humiliating feature that the author has failed to mention. Too frequently the notice for a letting is advertised in an engineering magazine of general circulation and it contains each and all of the points mentioned as being of interest to contractors. The result is that "out-of-town" contractors respond. They spend their time and money obtaining information and then submit a proposal which is sometimes the lowest bid. It is not a mere circumstance, nor is it evidence of fair dealing, when the low bidder is informed that "the work has been given to a local contractor". Under those circumstances, the engineer should withhold the advertisement from engineering magazines, and should state in the "Instructions to Bidders" that the contract will only be awarded to a local contractor.

Information for Bidders.—To obtain fair and equal competition between contractors, full and equal information should be given by the engineer to each and every interested contractor. Too many engineers have favorites. As a rule contractors are less "hidebound" than engineers, and they respond very quickly to fair and honest dealing.

On public work it is usually one of the regular requirements for the engineer to prepare an estimate in detail, and for private work it certainly has been prepared for the benefit and information of the owner. A responsible contractor obtaining the details contained in this estimate will not be governed or controlled thereby in his judgment as to bid prices. If the engineer is fairly competent to prepare the plans and specifications, he is the one most competent to portray the work involved on the plans by his estimate as to quantities, and this is the part of the detailed estimate in which responsible contractors are most interested.

A reasonable period of time between the date of the award of the contract and the date when actual work must be started by the successful contractor should be given. Involved estimates which have taken the engineer a long time to prepare generally also require at least a reasonable period of time on the part of the contractor before he can complete his negotiations with material and equipment dealers, sub-contractors, etc.

Awarding the Contract.—Bids for public as well as private work should be opened publicly and read. Secrecy plays no part in fair dealing or in the creation of an agreeable, straightforward, working relationship between engineer and contractor.

Certainly, in fairness to those who may be equally qualified by experience and financial responsibility, the contract should be awarded to the lowest bidder. Furthermore, when alternate items are included on the plans and in the proposal, the work should be awarded on the basis of the lowest alternate. When alternates are specified by the engineer, it is to be presumed, and should be only so considered, that each and all of them are equally adaptable for the work to be constructed. If this is not the case the engineer should declare himself clearly so as to avoid misleading the contractor and exciting the hostility of those directly interested in the alternate.

SPENCER A. SNOOK,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is timely and brings attention to carelessly worded specifications and to plans insufficiently detailed and designed. It also directs thought to the relation of surety bonds to contracts. It is desirable to have the honest co-operation of all parties and the co-ordination of all the factors involved.

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The writer believes that, in the majority of cases, the unit price bid is the best for all concerned. Changes which involve more than a simple reasonable increase or decrease of any item bid are sometimes necessary in the contract. Such changes are, or should be, beyond the control of the owner's engineer because of conditions not possible to foresee. Changes other than these should not be necessary, because plenty of time should be given to study the design, to prepare the drawings in sufficient detail, and to obtain the best borings and other sub-surface information. The engineer does not always have sufficient time to make complete plans; he is often forced to get them done within a certain time limit. Some engineers do not think it necessary to go into details, because they know that those who prepare shop drawings like plenty of leeway. This is a viewpoint that is favored altogether too much. With detailed plans available, the checking of shop drawings is made easier and quicker and the engineer is more likely to get what he really wants. Furthermore, engineers who know how to make plans with enough details will not do that work for the money customarily offered. It is the same old story—"if you want something good you must pay for it".

The specifications cannot be written too clearly and too concisely. Provision should be made for supplementary written agreements between the contracting parties for necessary and unusual changes. A description of the work to be done and possible obstacles to be overcome should be stated plainly.

To demand good workmanship alone might lead to considerable argument. If good results are specified, only good workmanship will produce them, generally speaking. However, when certain methods produce the good results desired, it seems best to call for those methods to be used, because the results otherwise may be considered good, but not just what was wanted. Also, it is important, very often, to call for materials of a certain kind, especially for concrete of a certain strength, or structural steel, bearings, etc., of a certain make, since good results cannot be judged by visual inspection.

The quantities bid upon should be calculated accurately to the best of the engineer's ability and information, since he knows the construction involved. The bidder should be furnished with all known information and requirements, and anything that is not positively known to be accurate should be so marked.

It would seem fair to ask the contractor to guarantee the work done under his control by means of a surety bond. There are some owners who may not know when a bid is too low or that the contractor is not a responsible one. Such people need some form of protection and for them a surety bond is desirable. The surety bond should be regarded as a guaranty against claims and damages and for the prompt execution of the work with the results specified, as all are within the contractor's control. If the owner never had any worries about these things, he would not require a contractor to obtain a surety bond. There must be some risk whenever a surety bond exists with a contract. The amount of that risk to be taken, respecting the responsibility of the contractor to pay for his labor and materials and to finish the work as desired, is something for the surety company to decide when writing its policy. If it takes good risks, the premiums should be low, but if it just wants to sell policies without knowing how responsible the contractor is, the premiums will be necessarily high.

Cash, or the bidder's check for bidding, is preferable to bid bonds. The acceptance of letters which guarantee a bond to the successful bidder is an undesirable practice and should be eliminated because of the possibility of collusion.

It might be well to keep the surety company advised on monthly estimates and changes in contract, having in view the ratio of work completed to that to be done within the given time limit.

J. A. Lenecek,* M. Am. Soc. C. E. (by letter).—The author has done a great service by offering a list of suggestions of a very practical value. It is regrettable, however, that this admirable paper suffers somewhat from the author's occasional criticism of the engineer's work through the different phases of competitive bidding. Sometimes these critical observations are self-contradictory.

For instance, the author admits that "nobody can see very far under water, or into the earth", but, later, he seems to forget that the word "nobody" should impartially include not only the contractor but the engineer as well. This inconsistency is apparent when the author suggests that the engineer should remove all uncertainties from sub-surface work, in advance of receiving bids. According to current practice the engineer supplies the bidder with all the necessary field facts (borings, soundings, etc.) and from these the contractor draws his own conclusion about the probable risk of the proposed work. No doubt an uncertainty always will exist if such field facts are not correctly interpreted by the bidder; but for such a condition the contractor should blame only himself, because it is primarily his affair to know his business. He should not expect any help from the engineer, who is usually busy in his own field. As a matter of fact the competent contractor knows how to analyze his costs correctly, while the inexperienced bidder, as a rule, estimates the risk too highly and on account of his high bid eliminates himself, which is a wholesome condition for the owner.

The author suggests a very good method of testing the bidder's financial responsibility, but it hardly offers any hint of how to test the contractor's competency. Perhaps the author also subscribes to the prevalent idea that a financially responsible contractor is in a position to acquire satisfactory plant and a good organization; however, money cannot buy successful experience and a good reputation.

In Spanish-American countries as well as in certain countries of Continental Europe, it is quite customary to subject the bidders on public work to a competency test. For this purpose a careful engineer's estimate is secretly prepared. Measures are also taken to prevent any information concerning the work from reaching the prospective bidders. To that end the chief engineer of the main Office for Public Works selects a few of his widely separated district offices and assigns to each office the preparation of a certain part of the engineer's estimate. The selection and assignments are known only to the chief engineer. Each district office presents its completed work in sealed envelopes at the public hearing held for the opening of the submitted bids. At this meeting all bids are tabulated. Later, the engineer's estimate is compiled from the sealed envelopes, and finally the successful bidder is selected. His detailed computations then must be submitted.

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The fortunate bidder is picked out in a manner quite different from general practice in the United States. The lowest bidder is not chosen, necessarily, but the one whose bid is numerically closest to the engineer's estimate. It is contended that by this method the contractor demonstrates the reliability of his experience and his organization and that he is qualified therefore to do the work. The bidder is also required to prove his financial responsibility. The writer has had an opportunity to observe that the entire system "works" remarkably well.

Jacob S. Langthorn,* M. Am. Soc. C. E. (by letter).—The author's long experience in the surety phase of contracting has enabled him to produce a very useful paper on the business side of engineering. A development of similar interest is the formation in 1927 of a Board of Reference in the City of Boston, Mass. This body is composed of twelve members, five selected by the Boston Society of Architects, five by the Boston Building Congress, and two by the Boston Society of Civil Engineers. It is aimed specifically to meet some of the injustices mentioned in the paper.

In service, the Board comes into operation in case a contractor finds items in plans or specifications that seem unfair or inadequate. He then refers the question confidentially to the Board before the competitive bids are submitted. The Board on its part will pass on the criticism and, if justified, will refer the question to the author of the plans and specifications. He, in turn, may answer the criticisms satisfactorily; otherwise, he will be asked to amend the forms or issue additional explanatory information to cover the omissions or defects. If time does not permit this, he may give assurance of future correction of defects. In case no agreement is reached, the Board will file formal reports with the three bodies from which it is formed. Such cases will constitute the basis for revisions in the Code of Practice which the organizations will be asked to approve. It is to be noted that there is no intention of acting in any way after contracts are once made.

As to Principle (9), it is exceedingly useful to have the date that a blue print is made shown on each copy. Especially is this important in building construction where the architect's tracings are made in pencil. Without the date it becomes difficult to establish priority.

In connection with Principle (14), one surety company to the writer's knowledge sends periodically a blank form following up work for which it has issued bonds. While the form does not require the complete monthly estimate, it does call for essential information as to the progress, percentage completed, and amount paid to contractor.

With Principle (29) there is a discussion of returning all documents with the bids. This practice is followed by many architects in New York

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City, but is manifestly unfair, as a contractor is entitled to retain the specifications and drawings upon which he has bid until his bid has been rejected.

Edward W. Bush,* M. Am. Soc. C. E. (by letter).—When preparing this paper the writer was frequently puzzled to know which points to include and which to omit. The general subject is broad and many important matters would have been included if the paper already had not stretched itself to a considerable length. Most of the topics are those on which the writer's experience indicated that changes could be made in the average current practice with economic advantages to the construction industry. A few subjects were included that, it is thought, would be of interest in the present-day discussion on the pre-qualification of bidders, the determination of the responsibility of a contractor, etc. These topics are closely interwoven in the general subject.

A few of the discussers have stated that the paper represented the viewpoint of the surety, but the writer disavows this and wishes it to be considered as the effort of an engineer who is drawing on many years of professional experience acquired before becoming connected with a surety company, as well as the experience gained after such connection was made. Some of the arguments and principles offered are adverse to what might be termed the best financial interests of the sureties.

The writer greatly appreciates the discussions offered. Many of them add supplemental material of distinct value, and all of them are of interest because they reflect the ideas of different persons variously engaged in the industry. Existing statutes will prevent the adoption at certain places of some of the principles even if they may be thought desirable and there will be many cases in which some of them are either not needed or inapplicable. A few of the discussions explain special conditions under which certain principles do not apply.

Mr. Wason points out that small scale drawings referred to in Principle (11) would be expensive and unsatisfactory, while photostatic reproductions would be simple and inexpensive. The writer termed them "reduced scale copies of the drawings" and had in mind the regular drawings reduced by some photographic process as is frequently done when they are bound with the contract.

In regard to Principle (12) Mr. Wason states that quantity surveys of this kind are sometimes used to adjust extras instead of merely as the basis for progress payments and such a use may do a rank injustice because the data may have been unbalanced. Whenever a contractor unbalances either a unit price bid or a schedule of the kind here considered, he takes a chance that later may work against him.

If the owner states in the contract (in accordance with the usual practice) that a certain percentage will be withheld from the progress payments, it is important to the owner that he observe this provision strictly and that he does not overpay the contractor at any time. Otherwise, he may be compelled to replenish the fund to the extent of the over-payment, for the benefit of those having lienable claims. Whether the method of paying for floor by floor, etc., suggested by Mr. Wason, is feasible without over-payment depends on the way the amounts are predetermined. Certainly, it is an easy way to handle a problem which, in the past, has been the subject of many acrimonious disputes betweeen contractors and owners.

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Principle (26) states that bids should be disposed of within thirty days after they are received. Of course, it would be desirable if this period were reduced to the ten days offered by Mr. Wason, but some public-work bodies meet only once a month; then, also, some time is frequently needed in which to finance a proposed construction. Therefore, thirty days were offered more as an outside figure than as a practice to be followed generally. The main thing is to state a time limit and to make this as short as possible.

Mr. Rudolph mentions the quantity-estimating agencies at various locations that now offer estimates to contractors and owners, and states that the engineer or architect who prepares the plans is better qualified to furnish such quantities than outsiders. Principle (28) suggesting quantity surveys applies only to building work and the writer had in mind quantity surveys of the kind offered by the owners in England. For some unknown reason the architects of the United States have never shown any enthusiasm for the English practice although contractors have many times pointed out to the owner the economic advantages if this practice were to become established. Engineers follow the practice every time a unit price contract is offered to bidders with the estimated quantities included. It is believed that the independent estimating agencies now operating in this country are the outgrowth of the efforts of groups of contractors to have joint or co-operative estimates prepared for them. A considerable volume of this quantity survey work is now being done—much more, perhaps, than many realize—and, of course, this labor really is being paid for by the owners. The writer agrees with Mr. Rudolph that this estimating can best be done, or at least supervised, by the architects who prepare the plans. If the volume increases and the agencies multiply, the architects, no doubt, finally will consider that quantity surveys are of some economic advantage and that their preparation is a matter to be supervised by them.

Mr. Wait's remarks are of especial interest as they are from a man who is broadly experienced in unraveling construction tangles. He questions, however, the value of the pre-qualification of bidders as to adequate financial strength and experience when he states that there are others not so fortunate nor successful who can carry out the contract, and that some of the biggest public works built to-day are by contractors who were not known to have financial backing or experience. Now and then one hears it said that "any contractor can get a bond no matter how weak financially, or inexperienced." Those behind the scenes, as it were, in the surety business know that such a statement is not true, and surety bonds on important contracts are not issued on behalf of inexperienced or financially weak contractors. Mr. Wait, perhaps inadvertently, gives the information that will reconcile his statement with the one just made by the writer when he says that "contractors who were not known to have financial backing" are able to handle large public work contracts successfully. No doubt, these contractors obtained surety bonds as public work was mentioned and if they were financially weak the sureties did not issue the bonds until after additional financial strength was put in the organizations. In most such cases, which are of common occurrence in surety underwriting, the fact that additional strength was put in is known to only the contractors, the sureties, and those supplying it. This is why some apparently weak contractors are able to obtain contract bonds.

Mr. Wait makes a valuable contribution to the paper when commenting on the absurdity of making the contractor assume the risk of the accuracy of the engineer's preliminary investigations of the site—work which may have cost a large sum. His remarks on the manner in which engineers allow lawyers to dictate what shall be put in the contract, are certainly refreshing.

Mr. White, in discussing Principle (13), questions the necessity for the owner to withhold a percentage of progress payments made to the contractor when the owner already has the protection of a contract bond guaranteeing the completion of the project. It is true that the bond extends this protection to the owner, but the present bond premium charges are based on the assumption that the customary 10 to 15% will be retained by the owner and if future practice materially reduces the retainage or discontinues it there is every probability that premium charges on contract bonds will be increased. There is a considerable volume of private work on which no bonds are required and the owners in such cases certainly need the protection of retainages.

The retainage often furnishes a good reason why the contractor rushes a job to completion and while it might be advantageous to the owner and the contractor in some cases to make all progress payments in full, it is believed that the time-honored custom of retaining from 10 to 15% is one that is generally desirable. A stipulation of full payments would require the monthly estimates to be made much more carefully than under the present practice of making approximate estimates when a retainage is withheld; and as the retainage is something earned by the contractor he can generally borrow against it if he needs the funds. In certain localities the owners permit the contractors to convert the retainage into high-class interest-bearing securities which are held by the owners, and this privilege has much to commend it.

Colonel Waldron, apparently, takes the position that the more the engineer tells the contractor in advance of bidding the higher the bids will be. He states that "when the contractor knows all hazards to be encountered, he is apt to bid higher than if he has to take a chance here and there." This assumes that the contractor is a rather stupid fellow who has to be told about a hazard in order to recognize it. It is believed that most present-day contractors are fully competent to recognize quickly the hazards and to bid accordingly. If they are not competent to do this, they do not remain long in the business of contracting. Principles (3), (6), and (25), with the supporting arguments, were offered in the paper with the hope that they would show that it is to the economic advantage of the owner to remove uncertainties if possible before offering work for bidding. As far as any interest of the surety on these points is concerned, it is well to keep in mind that the surety does not begin to pay on a claim until after the contractor has spent or pledged all he possesses. The contractor becomes a poor risk for himself before he is a poor risk for his surety.

Mr. White presents the contractor "as a human being entitled to justice" and points out that even after some of the uncertainties are removed the contractor has a strong economic factor to evaluate and a big risk to carry when he is engaged on contract work. In antithesis to this, Colonel Waldron states in the usual procedure "the engineer tries to get as much as he can for his employer; the contractor tries to do as little as possible in order to increase his earnings". The writer believes that most engineers and architects are fair-minded, with no desire to "put over" anything on a contractor; and, also, that most of the present-day contractors desire to perform their contracts fully. According to current business ethics it is poor policy for either to do the contrary. The greater proportion of all the construction work done in this country is in the hands of members of the Society, which will not knowingly admit a "crook", be he a contractor or an engineer.

Mr. Bernstein presents an interesting fact when he states that at times contractors on private work are compelled to accept paper in lieu of cash because the owners have failed to finance the undertakings completely. The writer has seen many financial statements of contractors showing large "accounts receivable" items which were uncollectable because the owners could not pay them. This is more apt to happen to a young contractor than to an experienced one because probably the latter has been caught, or is afraid he might be caught, in this way, and has learned to avoid such pitfalls. A somewhat similar topic was presented in the paper under the caption; "Introductory" and was mentioned in the paragraph preceding Principle (6).

In commenting on Principles (17) and (25), Mr. Bernstein points out that frequently the engineer lacks sufficient time to prepare complete drawings, etc., but that when sufficient time is available there is little cause for complaint. The observations of the writer, after being in a position to note this point on hundreds if not thousands of cases, leads him to state that there is room, when full time is available, for considerable improvement in the current practice on these details.

Mr. Stearns discusses many parts of the paper in an interesting and constructive way, but some comments on his discussion are needed. He mentions the advantage of the repetition in the contract documents of a subject like the provision that the general and all sub-contractors must obtain permits, etc. If Principle (11) is observed, the general conditions of a building contract always will be attached to the specifications of any particular trade and a requirement once expressed will apply to all trades.

In supporting his statement that certain "risks must and should be taken by the contractor", Mr. Stearns mentions a case in which longer piles were ordered than could be driven and as the number of linear feet driven was the item paid for, the cost of the extra lengths was borne by the contractor. This example is exactly of the kind the writer had in mind when writing Principles (3) and (6). In line with these principles, the owner in the example just mentioned would obtain more economic bids by dividing the pile work into two items, one covering the lengths delivered, and the other the lengths remaining below cut-off; also, by sub-dividing the first item and asking for bids on different probable lengths that will be required. In applying this scheme the engineer and contractor should confer on the probable lengths needed before the piling is ordered; then the contractor can prepare his bid with a reasonable assurance that he will not run into some unknown bottom condition that will cause him a loss and, therefore, can bid closely without a considerable factor added for the unknown. The writer has used this method of buying pile foundations and knows it to be successful.

Principles (7) and (8) were included by the writer because they are germane to Principles (18) and (19), and also because in recent years the industry has been offered some discussions on bonds which have overlooked many of the points mentioned in the paper. Mr. Stearns is correct in declaring that Principle (7) is a mere statement of fact—a fact that many States have overlooked when seeking to reduce the cost of contract bonds.

Regarding retainage mentioned by Mr. Stearns, the writer believes Principle (13) is true irrespective of whether or not a surety protection is obtained by the owner. Many border-line cases would not have been bonded by sureties if no retainages were held by the owners.

The writer fails to understand why an engineer or architect is frequently so reluctant to give the prospective bidder some idea in the advertisement as to the probable bid price. The engineer would tell the bidder verbally when he is starting his investigation or he would tell newspaper reporters preparing an article on the improvement. The bidder knows the size of job that will fit his organization and nobody is benefited when he bids against his best interests. Why lead him to investigate a $100 000 job when he is looking for one of $500 000, or *vice versa?* The sureties are constantly receiving many preliminary guesses on probable bid prices that are from 50 to 100% in error. Such guesses are disturbing factors in underwriting border-line contract bond cases and have caused much "grief" to both contractors and their sureties. The easy and obvious way is to tell the prospective bidder at the start the approximate price of the contract offered for bids.

Mr. Stearns does not believe the "bidding security" is in fact what the words imply. In nearly every case the contract documents precisely state the conditions under which the security is deposited—that it is a guaranty the contract will be executed, etc., if the bidder is awarded the contract. If the owner feels that he needs a "bidding security" he should get it large enough to cover the probable difference or spread between the bid that is accepted and the next higher even when these two bids are somewhat out of line. Taking Mr. Stearns' viewpoint of the matter, the 3 to 8% mentioned by him is not large enough to show sufficient working capital to finance the job and absorb an ordinary loss, especially as the contractor may have other contracts under way or may take subsequent contracts.

The writer is very much in accord with the closing paragraphs of Mr. Stearns' discussion and believes the paper could have included with propriety another principle on clarity and consistency. Many forms plainly show the confusion brought about from the "scissors and paste-pot" method of preparing them. The writer recalls a contract and specifications wherein the parties of the first and second part mentioned in the contract were transposed in the specifications so that among other things the owner was obligated to pay the contractor $25 per day for any delay in completing the contract beyond the date set.

Mr. Consoer offers many valuable suggestions among which are some on specific data that should be given to prospective bidders, which are in general agreement with Principles (3), (6), (17), (21), and (25). His practice of giving bidders a copy of the "Instructions to Inspectors" no doubt has prevented many disputes during the construction period and is one that could be followed with advantage to all parties.

Mr. Stephenson refers to Principle (21) and discusses whether or not an approximate estimate of the probable bid price, within limits, should be included in the advertisement. In connection with the "spread" between the high and low bids on an ordinary class of work and in cases where the highest bid is perhaps 50% greater than the lowest, the writer has generally considered that the top one-third or one-half of the bidders really did not expect to obtain the job but merely wanted to take it at very attractive prices. The discussion preceding Principle (30) considers this class of complimentary bidders and the disturbances they sometimes cause. Of the remaining bidders, a check-up of the volume of other work on hand, the financial condition, the previous experience on similar work, and whether or not new plant must be purchased and additional organization secured by the respective bidders, will show which of them really bid closely to obtain the job. In general, it is the bottom one-third of the bidders who, perhaps, are best equipped, who need the work, and are seriously competing for it. There is a variation to this that occurs now and then, wherein the talented contractors—those who really know what the job is worth—are found grouped considerably above the lowest bidders and it is evident that the latter are inexperienced and have made a sorry mess in their estimation of the price. Still another variation is that of the very capable contractor who, apparently, bid so far below the others that he faces a loss, but who at completion retains a fine profit after having given the owner a good job.

Professor Kirby offers a good summary of the general subject treated by the paper. His comment on Principle (39) questions whether the engineer should go far into the analysis of the contractor's financial condition, stating that this, perhaps, is within the scope of the surety's service. On work which is not bonded the engineer or architect will probably make the analysis if one is made, but on bonded work the surety will make the analysis, not as a service to the owner, but for its own protection. The writer included Principle (39) and its argument because they set forth briefly the methods of the sureties in determining the responsibility of contractors, and it was thought that this information would interest engineers and architects, especially in connection with the awarding of contracts that would not be bonded.

Mr. Root contributes an admirable discussion. The writer is in general accord with him and will only comment on one point. He states that small scale (reduced scale) drawings, bound with the contract, serve no useful purpose. The writer has reviewed a large number of cases in which the contract drawings, evidently reduced by a photographic process, were either bound with the contract or placed in an envelope attached to the back cover of the volume, and in no case has there been any uncertainty as to the details shown. Some of these jobs, running into millions, have been of large dams, bridge foundations, conduits, sewers, etc. The method will operate better with engineering than with architectural constructions, and the writer believes that if once tried the engineer will continue to use it on future work, if possible.

In the statement that "nobody can see very far under water, or into the earth", the writer attempted to epitomize many reasons why changes will be made in contracts after they are signed. The writer fully agrees with Mr. Lenecek that "nobody" includes the engineer as well as the contractor, but the writer does not recall that the suggestion is made anywhere in the paper that "the engineer should remove all uncertainties from sub-surface work". In Principle (3) and elsewhere the writer offers the thought that the more the uncertainties are removed, the lower will be the bids, which is quite different. Risks are inherent in certain kinds of work, but every time one of them is removed in advance of the bidding the price is lowered.

Mr. Lenecek states that the writer hardly offers any hint as to how to test the contractor's competency although a way to test his financial responsibility is given. In the discussion preceding Principle (39) the writer uses the word, "responsibility", as including, among other qualities, the financial strength and competency. To be responsible the writer states that in addition to being honest the contractor "should have ability as evidenced by his past experience and record; an organization suitable for the proposed work; and sufficient working capital or net quick assets to finance the operations and reasonable additional costs". Principle (33) refers to questionnaires and the footnote under "Introductory" mentions the "Standard Questionnaires" which were printed in the March, 1926, *Proceedings* of the Society. The writer knows of no working formula worthy of consideration that could be applied to the information secured on a contractor's past experience, etc., that would determine his competency. The matter, except that of financial strength, is one of individual judgment, because so many of the qualities, while capable of being recognized, cannot be evaluated. No two engineers or architects will give the same weight to the information, although it is to be expected that experienced engineers, architects, and surety underwriters will be found in fairly close agreement on an analysis of the kind of data herein considered.

The writer touches on a legal question here and there in the paper and wishes to state that all these points were checked by consultation with eminent legal counsel broadly experienced in the subject matter.

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SPECIFICATIONS FOR TRANSIT TRAVERSING AND STADIA LEVELING*

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With Discussion by Messrs. C. V. Hodgson, C. Kelsey Mathews, Howard S. Rappleye, and W. H. Rayner.

Synopsis

This paper has for its purpose a consideration of the effect of the errors in combination which attend the work of transit traversing and stadia leveling. It indicates a method whereby specifications for each operation in the field work may be prepared, so that the survey as a whole may be most economically and expeditiously executed.

The theory of errors has been adequately developed elsewhere, but for the most part, it has been applied to measures of precision, such as the probable error, and others, and to the adjustment of observations by the method of least squares. Authors of textbooks treat the various sources of error separately, but the total effect of the errors in combination in the surveying processes, appears to have received but little consideration. This is especially true as regards surveys of ordinary or low precision.

Many surveys are so simple in plan and are of such small scope that any detailed analysis of the methods would be absurd, but it has been the writer's observation that engineers who deal with surveys only occasionally, are apt to use about the same procedure and precision for one and all alike, with the result that on work of any magnitude or importance much time is wasted, either in unnecessarily refined measurements or in repeating the measurements to achieve satisfactory results.

It may be added that this paper is intended to indicate a method only, and is not a complete treatment of the subject. Many sources of error that are not mentioned will be found important under certain field conditions; but the method indicated herein should enable the engineer to specify intelligently—for any survey and under any conditions—the care which should be used for each part of the field work.

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DEFINITIONS

In the usual treatment of "systematic errors" it is assumed that, in some cases, their effect can be eliminated either by corrections applied to the measured quantity, or by systematic field methods. Thus, it is stated, the effect of the incorrect length of tape can be corrected by a computation, or the effect of non-parallelism between the axis of the level tube and the line of sight in an engineer's level may be eliminated by taking foresights and backsights of equal length. In this paper, however, a fundamental conception is that an error always is a quantity of which both the magnitude and the sign are unknown. Accordingly, in so far as the conditions mentioned do not affect final results, they are not considered as errors. In accordance with this conception, the following definitions are given.

A Systematic Error.—An error which always has the same (unknown) sign, and, therefore, is cumulative in its effect—but the magnitude of which cannot be calculated—is a systematic error. Its effect cannot be removed by systematic field methods.

Accidental Errors.—Such errors have the following characteristics (assuming that a large number of observations are taken): (*a*) Plus and minus errors are equally numerous; (*b*) small errors are more numerous than large ones; and (*c*) very large errors do not occur.

A Cumulate.—That part of the effect of a source of error which can always be eliminated either by a computation, or by a systematic field method, is termed a cumulate.

Mistakes.—Differences from true values, arising from confusion in the mind of the observer, are mistakes. They are detected by checking the work and are eliminated by applying corrections. It need not be added that the subject of mistakes has no place in a discussion of errors.

The Probable Error.—This is an error of such magnitude that in a series of like measurements, the number of errors larger, equals the number of errors smaller, than the probable error. In other words, in a series of like measurements, the probable error has such a value that the chances are even that the error in any single measurement, taken at random, is smaller (or larger) than the probable error.

These definitions are made clearer by the following examples.

Example 1.—Let it be supposed that a steel tape is to be used in a careful survey. It is sent to the U. S. Bureau of Standards, Washington, D. C., where its length is determined to be 100.021 ± x ft. In using the tape its length is assumed to be 100.000 ft. Hence, the quantity, 0.021, is the cumulate and the quantity, ± x, is the systematic error. In the process of chaining, the small errors made in marking the ends of the tape are accidental errors; and a mistake would be made if a fractional distance of 59.13 ft. was misread as 61.13 ft.

Example 2.—A level party is required to carry a line of levels across a river. The levelman reads a rod held on a bench-mark on the near bank of the river (B. M.$_1$), as 4.37 ± x ft., and on the far bank (B. M.$_2$) as 8.31 ± y ft., from which the difference in elevation is determined as 3.94 ft. He calculates the effect of curvature and refraction to be 0.04 ft. This is subtracted from the reading on B. M.$_2$ and the corrected difference in elevation is determined as 3.90 ft. In this case the quantity, 0.04 ft., is a cumulate; ± x and ± y are differences from true values affected by both kinds of errors: (1) The accidental errors arising from such causes as leveling the instrument, sighting, holding the rod, etc.; and (2) the systematic error due to the imperfect adjustment of the instrument.

Theory of Errors and the Inter-Relation of Errors

No attempt is made in this paper to derive the fundamental formulas.

Compensation.—This principle states that in a series of observations affected by an accidental error the total probable error is given by the product of the error in a single observation times the square root of the number of observations. Thus,

$$E_T = E_s \sqrt{n} \dots\dots\dots\dots (1)$$

in which, E_T is the total probable error; E_s is the probable error of a single observation; and n is the number of observations in the series.

The Summation of Errors.—This principle states that if the total probable error of a measurement is the result of several errors acting independently, the value of the total probable error is given by the square root of the sum of the squares of the separate probable errors arising from the several sources. Thus,

$$E_T = \sqrt{E_1^2 + E_2^2 + E_3^2 + \dots\dots E_n^2} \dots\dots\dots\dots (2)$$

in which, E_T is the total probable error, and E_1, E_2, etc., are the separate probable errors affecting the measurement independently.

The Product of Errors.—If a measured quantity, x, is the result of two independently measured quantities, A_1, A_2, such that $x = A_1 \times A_2$, then,

$$e_x = \sqrt{A_2^2\, e_1^2 + A_1^2\, e_2^2} \dots\dots\dots\dots (3)$$

in which, e_x is the error in the quantity, x; e_2 and e_1 represent the errors in the quantities, A_1 and A_2, respectively.

The more general form of this relationship is expressed in terms of the partial derivatives. Thus,

$$d_x = \sqrt{\left[e_1 \frac{\partial x}{\partial A_1}\right]^2 + \left[e_2 \frac{\partial x}{\partial A_2}\right]^2 + \dots\dots + \left[e_n \frac{\partial x}{\partial A_n}\right]^2} \dots\dots\dots (4)$$

Example 3.—The area of a rectangular city lot has been determined by the measurements: The length, $A_1 = 122.3 \pm 0.04$ ft., and the width, $A_2 = 48.3 \pm 0.03$ ft. What is the probable error in the area?

By Equation (3), since $x = A_1\, A_2$,

$$e_x = \sqrt{48.3^2 \times 0.04^2 + 122.3^2 \times 0.03^2}$$
$$= \pm 4.1 \text{ sq. ft.}$$

Example 4.—The height of a transit station has been computed from the measurement of a slope distance, $L = 837.3 \pm 0.2$ ft., and of a vertical angle, $\alpha = 3° 20' \pm 30''$. What is the probable error in the calculated elevation of Station A?

$$H = L \sin \alpha$$

and, therefore, by Equation (4),

$$e_H = \sqrt{e_L{}^2 \sin^2 \alpha + e_\alpha{}^2 \cos^2 \alpha \, L^2}$$

$$= \sqrt{0.2^2 \times 0.058^2 + 837^2 \times 0.00015^2 \times 0.998^2}$$

$$E_H = \pm 0.12 \text{ ft.}$$

Sources of Error in Chaining

The principal sources of error in chaining may be described briefly as follows:

(*a*) *Length of Tape, E_l.*—This source of error arises from the fact that a comparison made between a steel tape and a standard length, is not exact. A certificate given by the U. S. Bureau of Standards for a steel tape, is as follows:

"Bureau of Standards Certificate

"This certifies that 100-Foot Steel Tape No. 2432 has been compared with the standards of the United States and, for the intervals indicated, has the following corrections at 62° Fahrenheit when supported horizontally throughout its entire length under a Tension of 12 pounds avoirdupois:

Interval	*Correction*	*Probable Error*
(0 to 100 ft.)	— 0.002 foot	± 0.0003
Tension: 20 pounds (0 to 100 ft.)	+ 0.006 foot	"
Tension: 30 pounds (0 to 100 ft.)	+ 0.015 foot	"

"Note: The foregoing comparisons were made at the 0- and 100-foot lines on the brass sleeves.

"*A* + Correction means that the tape is too long, *i. e.*, longer than the nominal length of the interval; *a* — Correction that it is shorter by the amount indicated.

"The comparisons of this tape with the United States Bench Standard were made at a temperature of 83° Fahrenheit and in reducing to 62° Fahrenheit, the coefficient of expansion of the tape is assumed to be 0.0000062 per degree Fahrenheit."

While in use a 100-ft. steel tape is assumed to be exactly 100 ft. long; but reference to the certificate shows that for this tape its length at 62° Fahr. and under a tension of 12 lb., is 99.998 ft. ± 0.0003 ft. Hence, the cumulate is 0.002 ft. and the systematic error is ± 0.0003 ft.

(*b*) *Temperature, E_t.*—It will be noted that the U. S. Bureau of Standards certificate states the length of the tape at a temperature of 62° Fahr. and gives the value of the coefficient of expansion as 0.0000062 per degree Fahr. Hence, if the tape in the field has another temperature its length will be changed by an appreciable amount.

If the field temperature is not determined, its effect becomes a systematic error. If the value of the field temperature is determined, the change in length becomes a cumulate and may be eliminated by the proper computations.

However, because the temperature cannot be measured exactly, there will result a corresponding small systematic error in the length of the tape. If it is assumed that the temperature remains approximately constant for a given measurement, the errors resulting from temperature changes are systematic in kind.

(c) *Grade,* E_g.—Since the horizontal distances only are desired in taping, small errors will always be caused by the slope of the ground. This will be true whether the tape is stretched level or stretched on the ground because in the first case there will be a small deviation from a truly horizontal position and in the latter case there will be a small error in determining the inclination of the slope.

If the tape is stretched level, the resultant error will be systematic in kind because whether the tape is inclined upward or downward the resultant error will have the same sign. If the tape is stretched on the ground and if no correction is applied to the measurements, the errors (as in the case when the tape is not held level) will be systematic in kind. If, however, the inclination of the slope is determined, a cumulate is produced, and the corresponding correction can be applied. However, because of the errors in estimating the slope, small resultant errors will remain in the tape measurements, and because some estimated values of slope will be too great and some too small, the resultant errors will be accidental in kind.

Moreover, it will be noticed in Table 1 that a variation of, say, 1 ft., has different values for different slopes. For example, the variation between a 2% and a 3% slope is accompanied by a variation of 0.03 ft. in the error or cumulate; but the variation between a 6% and a 7% slope is accompanied by a variation of 0.07 ft. in the error or cumulate. This condition must be carefully observed in calculating the probable error due to slope of the ground.

(d) *Alignment,* E_o.—The effects of errors of alignment are exactly similar to those of grade, except that they can be minimized much more easily.

(e) *Sag,* E_s.—Whenever it is held off the ground a tape sags. For known conditions of pull, weight, and size of tape, cumulates may be computed and the corresponding corrections applied. Because of errors in determining the values of pull, weight, and size, small errors result. Since the variation in pull is the largest factor in causing the resultant error and because the amount of the pull will vary both above and below the estimated value, this resultant error will be accidental in kind.

If no correction for the effect of sag is applied, a systematic error results. Table 1 gives the values of the cumulates or errors for various conditions.

(f) *Marking Tape Lengths,* E_m.—The position of the end of the tape is usually marked with a stake, a pin, or kiel. Since this position can never be indicated exactly, and the distance thus indicated is as likely to be too great as too small, there result accidental errors the magnitude of which depend on the care used in marking.

(g) *Pull,* E_p.—A tape is elastic. It therefore has its correct length only under a given standard pull. For any other pull, the tape will be too long or

TABLE 1.—CUMULATES OR ERRORS IN TAPE MEASUREMENTS. 100-FOOT TAPE.

E_t, THERMAL EXPANSION.		E_p, STRETCH.		E_g, E_o GRADE, OR ALIGNMENT.		Weight of tape, in pounds.	E_s, SAG.				
							Pull, in Pounds.				
							Cumulate or error, in feet per 100 ft.				
Temperature change, in degrees.	Cumulate or error, in feet per 100 ft.	Pull, in pounds.	Cumulate or error, in feet per 100 ft.	Percentage of slope, or deviation from line.	Cumulate or error, in feet per 100 ft.		8.	10.	12.	15.	20.
7	0.005	7	0.005	1	0.005	1	0.065	0.041	0.029	0.019	0.012
15	0.01	15	0.01	2	0.02	1¼	0.092	0.058	0.041	0.025	0.014
22	0.015	22	0.015	3	0.05	1½	0.146	0.092	0.062	0.041	0.022
30	0.02	30	0.02	4	0.08	1¾	0.211	0.135	0.092	0.059	0.034
37	0.025	..	.....	5	0.12	2	0.260	0.167	0.110	0.073	0.041
45	0.03	..	.....	6	0.18	...	.....	.....	.....	.....	.....
52	0.035	..	.....	7	0.25	...	.....	.....	.....	.....	.....
60	0.04	..	.....	8	0.32	...	.....	.....	.....	.....	.....
67	0.045	..	.....	9	0.41	...	.....	.....	.....	.....	.....
75	0.05	..	.....	10	0.51	...	.....	.....	.....	.....	.....

too short and resultant accidental errors are caused. This effect of pull should not be confused with its effect on sag as considered under Article (*e*).

(*h*) *Wind*, E_w.—If a tape is stretched unsupported and a strong wind is blowing, the center of the tape will be carried to one side of the line joining the two ends of the tape. This condition causes an effect similar to, but usually much less than, that of sag.

SOURCES OF ERROR IN HORIZONTAL ANGLES

The principal sources of error in the measurement of horizontal angles are: (*a*) Centering the instrument over a point; (*b*) sighting a point; (*c*) reading the verniers; and (*d*) inaccurate adjustment of the instrument. These errors are mainly accidental in kind. A notable exception is the effect of non-perpendicularity between the line of sight and the horizontal axis of the instrument if, in traversing, the backsight is always taken with the telescope inverted, and the foresight is taken with the telescope normal, or *vice versa*. If this procedure is followed, a systematic error results. It can be largely eliminated and rendered an accidental error if at alternate stations the backsight is taken at the first station with the telescope inverted, and at the next station with the telescope normal. In careful work all the errors of non-adjustment are greatly minimized and rendered accidental in kind by double sighting, the telescope being reversed between sights.

EFFECTS OF SYSTEMATIC ERRORS IN TRAVERSING

Distances.—The effect of a systematic error in the length of the sides of a traverse is apparent in Fig. 1, in which, $A\ B\ C\ D\ E\ A$ represents the true figure of a given traverse. For an assumed error in the length of the tape it is supposed that the true distance, AB, is measured as AB', the error being BB'. Also, the true distance, BC, is measured as $B'\ C'$ and the total error at C is represented by the distance, CC'. The erroneous traverse, $AB'\ C'\ D'\ E'\ A$, is, therefore, similar in form to, and the sides are parallel with, those of the true figure. The total displacement of any point, as C, is given by the distance, CC'. This line is coincident in direction with, and proportional in length to, the line, CA, which may be called the radial line to the point of beginning, A. Likewise, the errors, DD' and EE', are coincident in direction with, and proportional in length to, the radials, DA and EA, respectively. Since the error, EE', is thus coincident with and proportional to the line, EA, evidently the error of closure at A is zero, which condition is plainly necessary if a systematic error is present throughout the traverse.

Angles.—The effects of the errors in the measurement of angles are accidental in kind, or they may be easily so rendered. Hence, the effect of systematic errors in angles will be disregarded.

EFFECTS OF ACCIDENTAL ERRORS IN TRAVERSING

Distances.—The total linear error resulting from the accidental errors of measurement of any given side of a traverse, is independent of similar errors in the other sides of the traverse. Accordingly, the total combined effect may be considered to be a function of the length of the traverse only; and since accidental errors are compensative in their effect, the total probable error resulting from the accidental errors of chaining will be given by the product, $E_L = e\sqrt{n}$, in which, E_L is the total probable error; e, the probable error of a single tape length; and n, the number of tape lengths in the traverse.

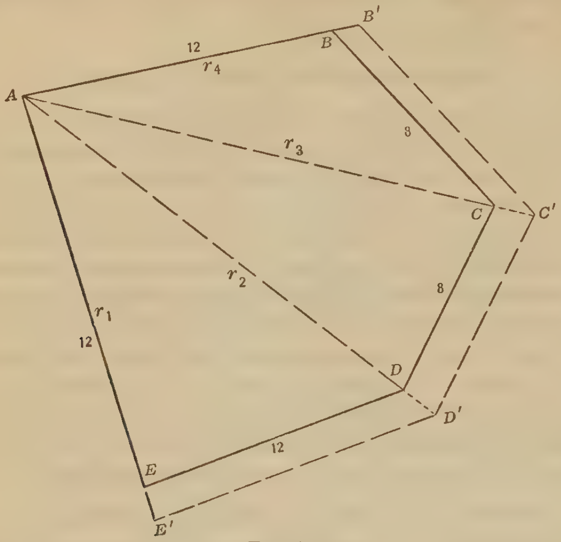

FIG. 1.

Angles.—Since the direction of each side of a traverse is determined by measuring the angle between it and the preceding side, it follows that an error made in measuring any given angle, affects all succeeding sides. This condition may be analyzed by reference to Fig. 1. An angular error at E will cause a linear error at A, the magnitude of which is given by the relation that, $e_1 = e_a \times EA$, in which, e_1 is the linear error at A; e_a, the angular error at E, expressed in radians; and EA, the length of the side, which may be expressed as r_1. Since 1′ of arc is very nearly 0.0003, in radians, then, if angles are read so that there is a possible error of 1′ in each one, $e_1 = 0.0003 \times r_1$.

Likewise, if the angular error at the station, D, is considered, the angle at E is now assumed to be without error, and an error in the direction of DE will displace the point, A, by an amount equal to the rotation of A about D through the angular error, e_a, at D. Hence, the linear error at A, due to angles only, will be given by the relation, $e_2 = e_a \times 0.0003\, r_2$, and, similarly, for the other stations, C and B. According to Equation (2) the total linear error at A will be,

$$E_A = \sqrt{e_a^2 \times 0.0003^2\,(r_1^2 + r_2^2 + \text{etc.})}$$
$$= 0.0003\, e_a \sqrt{r_1^2 + r_2^2 + \ldots + r_n^2} \ldots\ldots\ldots\ldots\ldots \quad (5)$$

SYSTEMATIC ERRORS IN AN OPEN TRAVERSE

Distances.—The total effect of the systematic errors in distances in the case of an open traverse has been indicated in describing the effect of these errors in a closed traverse. Thus, in Fig. 1, it has been shown that the total error at any point, as D, for example, is proportional to the radial length, DA. Accordingly, in Fig. 2, let the points, A, B, C, D, and E, represent the transit stations of an open traverse, and let the lengths of the sides, in 100-ft. stations, be shown by the numbers thereon. Thus, $AB = 600$ ft., etc. Hence, in this traverse, the total systematic error in distance will be proportional to the length, $EA = 3\,300$ ft.

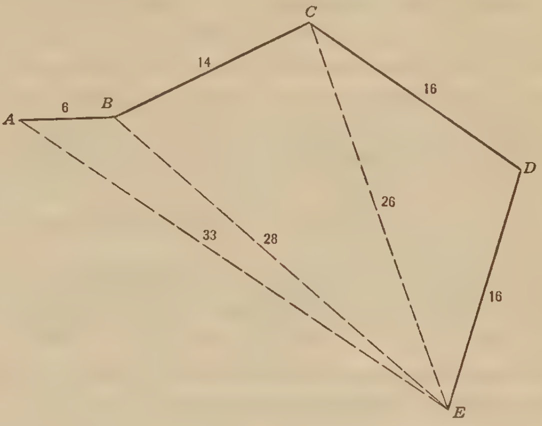

Fig. 2.

Angles.—The systematic errors in the measurement of angles will be disregarded in this paper.

Accidental Errors in an Open Traverse

The total effect of the accidental errors both in distances and in angles will be of a similar nature, and the probable errors are calculated in the same manner, as those for the closed traverse.

Applications of the Analysis to a Closed Traverse

Example 5 illustrates the application of this method of analysis to a closed traverse.

Example 5.—In Fig. 1, let the lengths of the sides be: $AB = 12$; $BC = 8$; $CD = 8$; $DE = 12$; and $EA = 12$ stations, respectively. The traverse was run under the following conditions: The tape was 100.03 ± 0.01 ft. at 62° Fahr. when supported throughout under a tension of 12 lb.; the estimated average temperature of the tape was 90° Fahr.; the weight of the tape was $1\frac{1}{2}$ lb.; the average ground slope was 5%; the country was partly open and partly wooden so that, for one-half the distance, the tape was held horizontal (unsupported) and, for the other half, it was held on the sloping ground; the average error in marking the tape lengths was ± 0.05 ft.; and the average error in measuring the angles was 01′. What is the calculated total probable error of closure?

Errors in Chaining.—The magnitude of the cumulates and the probable errors may be calculated as follows:

(E_l).—Assuming the tape to be compared with a standard tape, the length is found to be 100.03 ± 0.01 ft. at 62° Fahr. Hence, the cumulate is + 0.03 ft., and the systematic error is ± 0.01 ft.

(E_t).—It is assumed that the field temperature is estimated to be 90° Fahr. and that the probable error in this estimation is 15° Fahr. Hence, the cumulate (Table 1) is + 0.02 ft. and the systematic error is ± 0.01 ft.

(E_g).—If the average slope is 5%, and if the tape is held on the ground one-half the distance, then the cumulate (Table 1), is — 0.12 ft. per tape length. Also, if it is assumed that the probable error in estimating the slope is ± 1%, then the accidental error is found to be the difference between the cumulate for a 5% slope and that for a 6% (or a 4%) slope. These differences are found to be + 0.06 ft. and — 0.04 ft., or an average of ± 0.05 ft.

(E_s).—For a tape which weighs 1½ lb., and for a tension of 12 lb., the cumulate for sag is found (Table 1) to be — 0.06 ft. For an assumed variation of ± 3 lb. in the tension, the change in the effect of sag is — 0.03 ft. and + 0.02 ft., or, say, ± 0.03 ft.

(E_p, E_o).—These effects are assumed to be negligible.

(E_m).—If the probable error in marking the tape lengths is ± 0.1 ft., this effect is an accidental error. No cumulate can be calculated for this source.

The combined effect of these separate errors is found by Equation (2), and the computations are conveniently arranged as shown in Table 2.

TABLE 2.—COMPUTATIONS TO DETERMINE CHAINING ERRORS IN A CLOSED TRAVERSE.

Source.	Cumulate.	Error.	Kind.	Amount, q.	q^2.
E_l	$0.03 \times 52 = + \ 1.6$	±0.01	Systematic	0	......
E_t	$0.02 \times 52 = + \ 1.0$	±0.01	Systematic	0	......
E_g	$0.12 \times 26 = - \ 3.1$	±0.05	Accidental	$0.05 \sqrt{26} = \pm 0.3$	0.09
E_s	$0.06 \times 26 = - \ 1.6$	±0.03	Accidental	$0.03 \sqrt{25} = \pm 0.15$	0.02
E_p	Neglect	......	......	......	......
E_o	Neglect	......	......	......	......
E_m	0	±0.1	Accidental	$0.1 \sqrt{52} = \pm 0.7$	0.49
Σq^2	......	......	......	......	0.61

Total correction = — 6.6 ft.; total $E = \sqrt{0.61} = \pm 0.8$ ft.

Errors in Angles.—According to Equation (5), the total probable linear error of closure resulting from the accidental errors of measuring the angles is given by the relation:

$$E_A = 1 \times 0.0003 \sqrt{1\,200^2 + 2\,000^2 + 2\,000^2 + 1\,200^2} = \pm 1.0 \text{ ft.}$$

Total Probable Error of Closure.—Combining the probable errors for distance and angles, the total probable error resulting from both sources is,

$$E_T = \sqrt{E_L^2 + E_A^2} = \sqrt{1.0^2 + 0.8^2} = \pm 1.3 \text{ ft.}$$

This error of closure (about 1 in 4 000) may seem surprisingly small for the field conditions named. It indicates three important facts: First, that systematic errors in distances and in angles (provided they are present throughout the work) do not affect the error of closure; second, that the total errors of traversing can be easily and largely reduced if the cumulates are removed by simple calculations or by systematic field methods; and, third, that the computations are based on the assumption that the errors are accidental in kind. If any sources of error operate as systematic errors through a part and not all of the work, then the error of closure will be seriously affected thereby.

Application of Analysis to an Open Traverse

Errors in Chaining.—Let Fig. 2 represent an open traverse for which the same field conditions apply as those assumed for the closed traverse of the previous case.

It has been shown that the systematic errors in chaining from one point to another are proportional to the length of the line connecting these points.

The cumulates will be the same as those calculated for the closed traverse and the total probable error in chaining is calculated as shown in Table 3.

TABLE 3.—Computations to Determine Chaining Errors in an Open Traverse.

Source.	Kind.	Amount, q.	q^2.
E_l	Systematic	$0.01 \times 33 = \pm 0.33$	0.11
E_t	Systematic	$0.01 \times 33 = \pm 0.33$	0.11
E_g	Accidental	$0.05 \times \sqrt{26} = \pm 0.26$	0.09
E_s	Accidental	$0.03 \times \sqrt{26} = \pm 0.15$	0.03
E_p		Neglect	....
E_o		Neglect	....
E_m	Accidental	$0.1 \times \sqrt{52} = \pm 0.7$	0.49
$\Sigma_{q^2} =$			0.84

Total $E_L = \pm 0.9$ ft.

Errors in Angles.—The final error resulting from accidental errors in measuring the angles is calculated by Equation (2) as follows:

$$E_A = 1 \times 0.0003 \sqrt{2\,800^2 + 2\,600^2 + 1\,600^2} = \pm 1.2 \text{ ft.}$$

Errors in Chaining and in Angles Combined.—The combined effect of the errors in chaining and in angles may be represented by E_T and, therefore,

$$E_T = \sqrt{E_L^2 + E_A^2} = \sqrt{0.9^2 + 1.2^2} = \pm 1.5 \text{ ft.}$$

It may be noted that this traverse has the same length as that of the closed traverse, and the difference in magnitude between the errors of closure of ±1.3 ft. for the closed, and ±1.5 ft. for the open, traverse is caused entirely by systematic errors, operative in both cases. These were eliminated in the closed traverse, but could not be eliminated in the open traverse. This condition calls attention to the necessity of giving careful attention to sources of systematic errors for traverses which do not close, if the errors are not to exceed prescribed limits.

PREPARING SPECIFICATIONS

General Procedure.—In the preceding pages a method has been indicated whereby the total probable error of displacement of the terminal point of a traverse can be calculated if the field conditions are known. A principal purpose of this paper, however, is to indicate the reverse procedure; that is, for a known permissible error of displacement of the terminal point of a traverse, to write specifications for the field work such as will yield the desired results. Assuming that the permissible total error of displacement and the field conditions are known, the successive steps in the procedure are, as follows:

(*a*) Assign the total permissible error to the two general sources of error, namely, those of angles and distance, respectively.

(*b*) Assign values to the separate sources of error in the chaining such that their combined effect will not exceed the amount assigned to distance under Step (*a*).

(*c*) Calculate the permissible angular error for each transit station such that the combined effect will not exceed the amount assigned to angles under Step (*a*).

(*d*) Write the specifications to govern the field work in accordance with the values found under Steps (*b*) and (*c*).

Example 6.—An open transit-tape traverse approximately 1 mile long is to be run with a total permissible error of displacement of 1.0 ft. The ground is rolling with an average slope of about 6%; it is partly wooded and partly open, so that for one-half the distance the tape is held unsupported and for the other half it is held on the sloping ground; the shape of the traverse is sketched as shown in Fig. 2. It is required to write specifications for the field work involved:

(*a*) For this traverse an equal amount of permissible error will be assigned to distance and to angles. Accordingly, by Equation (2),

$$1 \text{ ft.} = \sqrt{E_L^2 + E_A^2}$$

or,

$$1 \text{ ft.} = 2\, E_L^2$$

hence,

$$E_L = E_A = \pm\, 0.7 \text{ ft.}$$

(*b*) By arranging the computations as shown in Table 4, an estimate of the distribution of various chaining errors is obtained.

(*c*) It is assumed that systematic errors in measuring angles will be eliminated in the field and the permissible error will be calculated for accidental errors only. The values of the radial distances, r, are estimated; hence,

$$E_A = \pm\, 0.6 \text{ ft.} = e_a \sqrt{2\,800^2 + 2\,600^2 + 1\,600^2}$$

$$e_a = \pm\, \frac{0.6}{4\,100} = 0.00015 = \pm\, 30''$$

(*d*) According to the values assigned to the various sources of error under Steps (*b*) and (*c*), the following specifications will apply to the field work:

The length of the tape (E_l) shall be known within ± 0.005 ft.

The temperature of the tape (E_t) shall be determined within 15° Fahr.

The slope of the ground (E_g) shall be estimated correctly within an average error of 1 per cent.

The tension on the tape (E_s) to compensate for sag (weight, 1½ lb.) shall not vary from the standard tension (12 lb.) by an average error greater than 3 lb.

TABLE 4.—COMPUTATIONS FOR DETERMINING CHAINING ERRORS IN EXAMPLE 6.

Source.	Kind.	Amount, q.	q^2.
E_l	Systematic	$0.005 \times 33 = \pm 0.16$	0.03
E_t	Systematic	$0.01 \times 33 = \pm 0.33$	0.11
E_g	Accidental	$0.06 \times \sqrt{26} = \pm 0.30$	0.09
E_s	Accidental	$0.03 \times \sqrt{26} = \pm 0.15$	0.03
E_p	Neglect	....	....
E_o	Neglect	....	....
E_m	Accidental	$0.05 \times \sqrt{52} = \pm 0.35$	0.12
$\Sigma_{q^2} =$			0.38

$$E_L = \pm 0.6 \text{ ft.}$$

Errors due to variations in pull (E_p) may be neglected.

Errors due to variations from true alignment (E_o) may be neglected.

The positions of the ends of the tape shall be marked in such a manner that the average error (E_m) shall not exceed ± 0.05 ft.

The average error (E_a) in measuring the angles shall not exceed ± 30″.

STADIA MEASUREMENTS

Analysis.—The measurement of horizontal distances by the stadia method is affected by the systematic errors in the determination of the interval factor, K, differential refraction, inclination of the rod, etc., and by the accidental errors of reading the rod intercept, S, the vertical angle, α, etc. The effects of these errors, both for closed and for open traverses, will be similar in nature to those described for chaining.

The stadia formula for the horizontal distance is,

$$L = K\,S\cos^2\alpha + (F + c)\cos\alpha \qquad (6)$$

in which, L is the distance; S, the rod intercept; K, the factor interval; α, the vertical angle; F, the principal focal length of the objective lens; and c, the distance from the objective lens to the vertical axis of the instrument. The value, $(F + c)\cos\alpha$, is negligible and will be disregarded.

According to Equation (4),

$$E_L = \sqrt{(e_k\,S\cos^2\alpha \times N)^2 + (\sqrt{N}\,K\,e_s\cos^2\alpha)^2 + (\sqrt{N}\,K\,S\,e_a\sin 2\,\alpha)^2}$$

in which, E_L is the total probable error, and N is the number of stadia readings.

Specifications.—The specifications for any transit-stadia traverse may be prepared by the following procedure (see Steps (*a*) to (*d*), page 690): (1) Decide on the total permissible error; (2) assume average values for S and α and the probable errors for K, S, and α; namely, e_k, e_s, and e_a; (3) compute the total probable error; (4) revise the assumed values of the probable errors, if necessary, to meet the required degree of accuracy; and (5) write specifications for field work.

Example 7.—It is desired to measure the length of the traverse shown in Fig. 2 by the stadia method such that the probable error of displacement of the point, E, shall not exceed ± 10 ft. Write specifications for the field work:

(1) The permissible probable error is ± 10 ft.

(2) The average values are assumed to be $S = 5.00$ ft.; $\alpha = 3°$; $e_k = \pm 0.3$; $e_s = \pm 0.03$ ft.; and $e_\alpha = \pm 02'$.

(3) In this example the error, e_k, is a systematic error, and e_α and e_s are accidental errors. Since the first term under the radical represents the systematic error, n, for this term, will be $\frac{33}{5} = 7$, and for the second and third terms, n will be $\frac{52}{5} = 10$. Hence,

$$E_L = \sqrt{(0.3 \times 5.00 \times 0.999^2 \times 7)^2 + (\sqrt{10} \times 100 \times 0.03 \times 0.999^2)^2 + (\sqrt{10} \times 100 \times 5.00 \times 0.0006 \times 0.11)^2}$$

$$= \pm \sqrt{10^2 + 9^2 + 0.1^2} = \pm 14 \text{ ft.}$$

(4) The calculated value, ± 14 ft., is greater than the permissible amount of ± 10 ft. It is noticed that the largest share of the total error is caused by the error, e_k; hence, it is assumed that by a more careful determination this is reduced to ± 0.1 ft.; it is also noticed that e_α yields only a very small error, and it will expedite the field work to increase this permissible error to ± 5′. Using the revised values,

$$E_L = \pm \sqrt{3.3^2 + 9^2 + 0.3^2} = \pm 9.5 \text{ ft.}$$

(5) The specifications for the field work are:

(*a*) The stadia factor interval shall be determined within a maximum error of ± 0.1.

(*b*) The average error in reading the rod intercept shall not exceed ± 0.03 ft.

(*c*) The average error in reading the vertical angles shall not exceed ± 05′.

Remarks.—The results determined in Example 7 show that the accuracy of stadia measurements is increased by reducing the systematic errors, and that, for small vertical angles, the effect of the error, e_α, is quite negligible. For large angles, however, the analysis gives a method for determining the required degree of precision in all factors.

STADIA LEVELING

The procedure for preparing specifications for the work of stadia leveling is the same as for stadia traversing.

Example 8.—It is desired to run a line of stadia levels 5 miles long with a permissible error of closure of ± 3.0 ft. Write specifications for the field work:

(*a*) The permissible error of closure is ± 3.0 ft.

(*b*) Since by the methods used in stadia leveling all errors become accidental in kind and, therefore, compensative, the average permissible error for a single reading may be calculated. Thus,

$$e_s = \frac{E_t}{\sqrt{n}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which, e_s is the probable error of a single reading; E_t, the total probable error; and n, the number of observations. For a traverse 5 miles long and for average sights of 400 ft., the number of sights will be about 65. Hence,

$$e_s = \frac{3.0}{\sqrt{65}} = \pm\ 0.37 \text{ ft.}$$

(*c*) Assume the stadia constant, K, to be known with a probable error of ± 0.3 ft.; the rod intercept to be read with a probable error of ± 0.03 ft.; and the vertical angle to be measured with a probable error of ± 02′. Accordingly, $e_k = \pm\ 0.3$ ft.; $e_s = \pm\ 0.03$ ft.; and $e_a = \pm\ 02'$.

The stadia formula for the difference in elevation, V, is given by the relation,

$$V = \frac{1}{2}\ K\ S \sin 2\ \alpha + (F + c) \sin \alpha$$

The quantity, $(F + c) \sin \alpha$, will be neglected in the following calculations.

(*d*) By Equation (4) and by assuming the average rod intercept to be 4.00 ft., and the average vertical angle, α, to be 3°:

$$e_s = \sqrt{\left(\frac{1}{2}\ e_k \times s \times \sin 2\ \alpha\right)^2 + \left(\frac{1}{2}\ K\ e_s \sin 2\ \alpha\right)^2 + \left(\frac{1}{2}\ K\ s\ e_a\ 2 \cos \alpha\right)^2}$$

$$= \sqrt{\left(\frac{1}{2} \times 0.3 \times 4.00 \times 0.11\right)^2 + \left(\frac{1}{2} \times 100 \times 0.03 \times 0.11\right)^2 + \left(\frac{1}{2} \times 100 \times 4.00 \times 2 \times 0.0006 \times 0.99\right)^2}$$

$$= \sqrt{0.07^2 + 0.17^2 + 0.24^2} = \pm\ 0.30$$

It is seen that the assumptions in Step (*d*) yield a probable error, $e_s = \pm\ 0.30$ ft., which is less than that required under Step (*b*). Accordingly, the probable error in reading the rod may be increased to ± 0.04 ft. Using this value, the total probable error, e_s, equals ± 0.33 ft., which is satisfactory.

(*e*) The specifications for the field work are:

1.—The stadia constant, K, shall be determined with a probable error not exceeding ± 0.3.

2.—The probable error in reading the rod shall not exceed ± 0.04 ft.

3.—The probable error in reading the vertical angles shall not exceed ± 02′.

CONCLUSION

The actual behavior of errors agrees more nearly with their theoretical characteristics the more carefully the measurements are made. In other words, if the conditions under which the measurements are made, are carefully controlled, the systematic and accidental errors can be more perfectly classified than if the control of the measurements is less complete. Accordingly, theoretical and actual results, as regards the errors in surveying measurements, will agree more closely under the former than under the latter conditions. However, for the relatively crude measurements of low precision it is believed that the speed of the work can be increased and the character of the results greatly improved by the judicious use of the methods here presented.

DISCUSSION

C. V. Hodgson,* M. Am. Soc. C. E. (by letter).—The author has outlined a logical method of determining the specifications for accuracy in a survey and goes further by presenting the mechanics of designing them. The method shown is not only logical, but it accords well with engineering principles. In designing a bridge the engineer must know the purpose to be served, the load to be provided for, and then must decide what type of construction will best meet those requirements. The detailed designing can then be begun and the strength of the individual members computed. Similarly, in a survey, the purpose must first be clearly understood, the requirements of accuracy and specialized data to be met must be formulated, and then decision as to the type of survey and kind of instruments to be used, can be made. The specifications for the strength, or accuracy, of the individual operations must then be determined.

Although the method of writing specifications for a survey described by the author is not new in principle, it shows in a clear manner the method of combining the errors attributable to each of a number of associated factors in a surveying operation and emphasizes the important distinction between a systematic and an accidental error. This distinction is the most essential factor to be borne in mind by the surveyor, except for the necessity of adopting methods which will avoid the possibility of mistakes or blunders remaining undetected. One wishes that the sentence in Professor Rayner's definition of "Mistakes" were always true—that "they are detected by checking the work and are eliminated by applying corrections." Despite elaborate systems of checking, the chances for mistakes have been found by all surveyors to be surprisingly numerous, and there is usually the possibility that some blunder will not be detected by the field checks or by the office computation.

The examples given in the paper show strikingly the difference in the rates at which systematic errors and accidental errors accumulate. A systematic error of ±0.1 ft. per tape length with a 100-ft. tape, will give an error of 1 part in 1 000, and this error is independent of the number of tape lengths. An accidental error of +0.1 ft. per tape length, in 100 tape lengths, will represent a probable error of ±1.0 ft., or 1 part in 10 000. The principal aim of the surveyor in securing a specified accuracy is to reduce systematic errors to a permissible limit by his methods of measurement, and to make certain that what he regards as accidental errors do not tend to become systematic ones. For instance, in Table 4, the estimated tension is taken to vary from the standard tension by a probable error not greater than 3 lb. and this is considered to be an accidental error. However, if the estimation of tension should be consistently higher or lower than the standard tension, as would conceivably be the case in actual practice, a systematic error would be introduced which would accumulate at a different rate than an accidental error. The same principle would apply to the estimation of temperatures or slope of ground.

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It is probable that some may question the adequacy of the definition given for "Error", which states that "an error always is a quantity of which both the magnitude and the sign are unknown." That part of the definition relating to the sign does not seem to be correct or essential, because the error due to the wind effect of a suspended tape is always such as to make the measured length too large, as is also the error due to lack of alignment of a tape. Other examples of the same tenor could be cited. Also, many will prefer a definition for "Systematic Error" which will provide that it need not be always of the same (unknown) sign, but instead that it shall follow some definite law or relation. For instance, the temperature error in a series of tape measurements may conceivably cause both minus and plus errors at different parts of the same series of measurements, because the temperature varies above and below that at which the tape is standard, but since the error varies in accordance with some definite law, it may be called systematic.

C. Kelsey Mathews,* Assoc. M. Am. Soc. C. E. (by letter).—The author has presented this topic in a manner that should appeal to the engineering mind, inasmuch as the subject of specification writing for the assurance of proper results by an economic process constitutes a major part of the design of all engineering projects. Engineers have been more or less altruistic in attempting to obtain the ultimate economy for the client through refinements in design and construction and have neglected to apply the same principles of economy in their own work. The principles set forth in the paper may be applied in a practical way to reduce expenses.

A large part of the expense of an engineering business may be for field surveys. Generally, the conception of the project originates with some one man in the organization, and through him is determined what field information is to be obtained. The manner in which this is to be achieved is generally left to an assistant whose only qualification for directing surveys may be several years' experience in manipulating surveying instruments.

The lack of knowledge of the purpose of the various parts of the survey, the various degrees of precision required, and the methods necessary to secure this precision, generally results in financial loss to the business. The principal in charge of a project should have a thorough knowledge of these factors, and detailed "specifications", either written or oral, depending on the scope of the project, should be given the assistant in charge of surveys, so that the required precision may be obtained at the least cost. In this manner only the manipulation of instruments is left to the field men.

The errors in such manipulation are those of observation and judgment; they are "accidental" in character, and the effect on the result is comparatively small if the surveyors are skilled and experienced. The important factors will be disposed of as "systematic errors and cumulates" by the observance of the requirements of the "specifications". Provision should be made in the specifications for proper methods of checking against mistakes.

There are two obstacles to be overcome in profiting by specification writing for surveys. The first and most important is the ignorance of the "principal" as to the existence of the various sources of error. The second is the reticence of self-styled surveyors to observe new ideas which are not consistent with habits acquired through years of manipulating instruments.

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The author's analysis and definitions should provide a liberal education, tending to overcome the first obstacle, if the "principal" can be convinced of the economy which will result. The detailed analysis set forth will actually be a post-graduate course to many engineers after learning that a transit may have a "stadia factor interval" different from 100, and that such difference is of much more importance than exceeding care in reading the intercept. In one out of twenty transits of various makes and ages, the writer found that ten had a factor of 100; seven, of 99 or 101; two, of 102; and one, of 105.

The writer knows of engineers who have insisted that plans and specifications shall obtain consistent economy for the client; who, nevertheless, have taken no cognizance of the possibly large cumulative error involved in the stadia interval. At the same time, they read horizontal angles in the same traverse with a magnifying glass on a 1-min. transit.

One topographical survey for a large hydro-electric project, which kept four parties in the field for six months, was made with no other important instructions than to "get two miles of traverse per day". Several days were lost in one instance in computing and re-surveying one traverse, run with two transits, which failed to close "satisfactorily", only to find later that one of the transits was "out of adjustment"; that is, the interval factor was not 100 as assumed. Numerous examples of such uneconomical procedure may be cited, all of which might be eliminated by "specifications".

The application of the laws of accidental errors to surveying measurements and to the propagation of errors through a combination of different measurements and calculations is certainly more reasonable and consistent than its application to some other engineering data, such as flood flows. This is especially true considering the limited records available for most engineering calculations.

The probability curve is determined theoretically only by an infinite number of observations. In determining the probable frequency of a certain maximum flood, based on a limited number of observations which are under no definite control, the truth is only confused by a mass of probability figures. In shooting at a mark, or in estimating the slopes of the various courses of a traverse, the observations are under control, and the deviations from true value will be truly accidental in character. The laws of probability, therefore, will apply with greater mathematical certainty even if the observations are few in number.

The value of specification writing for transit traversing, stadia leveling, or any other form of surveying, can be realized only if the field men understand and comply with the specifications. The wording and the minuteness of detail necessary will depend on the type of men for whom they are written.

The writer believes that the "specifications" as outlined by the author in the examples given should be classed as calculations and notes made by and for the principal in charge as a guide for writing the actual specifications. These calculations and notes might be given the assistant in charge of field work so that the spirit and purpose of the actual specifications might be more easily understood. The knowledge that partial derivatives and a few square roots had been introduced might be confusing to some "practical" surveyors and might not secure the co-operation desired.

Consider, for instance, the data given in Example 6. The eight specifications listed under Section (*d*) are the principal's notes. The suggested working specifications to be handed to the field assistant would read as follows:

(1) Compare the tape with standard by stretching between hubs (set permanently near office) at a pull of 12 lb. determined by a small spring balance. Note correction in length to nearest 0.01 ft. and note air temperature from the thermometer to nearest degree, before leaving office.

(2) In the field, estimate temperature of tape at beginning and completion of chaining and note to nearest 10 degrees.

(3) Estimate average slope by aid of hand level for each critical change in the slope and note to nearest 1 per cent.

(4) and (5) Hold chain parallel to the general slope of the course, or on the ground, maintaining as nearly as possible, by estimation, a 12-lb. pull.

(6) Set flag on forward station and have rear chainman align head chainman therewith when deviation is apparently more than 1 ft.

(7) Stick pins or mark, without elaborate care, so that the station is marked within 0.05 ft. of apparent exact position.

(8) Use 1-min. transit; measure all angles to right with telescope direct; read and record angle to nearest minute without magnifying glass; reverse telescope, reset plates, and read and record angle to right again; read needle and record to nearest degree.

Checks.—The field work may be checked as follows: (1) In taping, both chainman shall calculate all stations from pin count and call and check; (2) both chainmen shall call and check all pluses; (3) in measuring angles, duplicate all readings; and (4) read the compass.

Specifications for calculating and removing cumulates should be written, or, preferably a blank form should be provided for entering the corrections with the proper sign, and the corrected chaining.

The sixth specification under Section (*d*) stipulates that, "errors due to variations from true alignment (E_0) may be neglected". The writer has included an item (corresponding to Example 6) that controls the precision desirable for this source. The author has stated in Item (*d*), under "Sources of Error in Chaining", that, "the effects of errors of alignment are exactly similar to those of grade, except that they can be minimized much more easily." The writer recognizes the error due to alignment in the sense of chaining along a curved line between two traverse hubs rather than along a straight line, and acknowledges that this error may be neglected if a sincere attempt is made to chain toward the forward hub. The source of error which should be controlled is the "zigzag" deviation of the ends of the tape from the general line of chaining independent of its general curvature. The first error is similar to that introduced by chaining parallel to, or on, the general ground slope without correcting to horizontal. This error would amount approximately to 0.1 ft. in Example 6 if the average offset from a straight line between hubs were 3 ft. The second error is similar to that introduced by the ends of the tape not being at the same elevation in attempting to measure horizontal distance directly, or by not being in a plane parallel to the general slope in chaining according to the problem assumptions. This error would amount approximately to 0.25 ft. in Example 6 if the average "zigzag" deviation of the end of tape from the general line of chaining were 1 ft.

An interesting point is developed in connection with this subject when a comparison of the precision obtainable by chaining on the ground and removing the slope cumulate is made with that obtainable by attempting to hold the tape horizontally and set the point by plumb-bob.

If the grade is estimated for every material change in the general slope along a traverse with an average error of 1% (which may be done quickly with the aid of a hand level), the error in the corrected length of traverse will be "accidental" and will amount to the square root of the number of estimates times the average error in length per estimate. In Example 6, if there are 25 estimations of slope (1 per station), the probable error due to estimating grade would be $0.06 \text{ ft.} \times \sqrt{0.25} = \pm 0.30$ ft.

The author has assumed that, for the remaining half of the traverse, the tape is held unsupported and horizontally, but has allowed nothing for the error introduced because of the ends of the tape not being in the same horizontal plane. This error would be 33×0.005 ft. $= 0.17$ ft., if the deviation from horizontal were 1 per cent. If the average slope were 3%, or less, the precision obtainable by the two methods would be nearly the same, with the time advantage in favor of chaining on the ground or parallel to the general slope.

The determination of the stadia constant within a probable error of ± 0.1 is rather unusual, and the method of obtaining it should be outlined in detail for field specifications. These might read as follows, for Example 7:

> "Ten arbitrary points on level ground and in line from the transit shall be marked up to approximately 1 000 ft. The intercept shall be read to the nearest 0.01 ft. on each distance. The distance to each shall then be taped and recorded to the nearest foot. The value of K for each distance shall then be calculated from the formula, $K = \left(\frac{D - 1}{S}\right)$, and recorded to the nearest 0.1. The mean of the ten values shall be calculated to the nearest four significant figures and used for K."

These suggested specifications for determining K are calculated from an assumed probable error of ± 0.003 ft. in reading the intercept and of ± 0.3 ft. in distance, which is equivalent to a probable error of ± 0.3 ft. in K. The probable error in the mean of ten determinations would then be approximately ± 0.1 ft. The probable error in K could be checked more accurately by computing the residuals for each determination afterward.

It is necessary to have the proper perspective with regard to the surveys for a project, in the same sense as a proper perspective is necessary in the conception of the project as a whole. In other words, an economical survey cannot be made by an unrelated system of measurements any more than a structure can be built economically by adding arbitrary units to a nucleus. The designer must visualize the component parts assembled, and must know their correlation.

The author is to be congratulated on presenting a paper containing the fundamental principles necessary to attain the proper perspective with regard to the surveys, and also practical suggestions for their direction and execution.

Howard S. Rappleye,[*] Assoc. M. Am. Soc. C. E. (by letter).—This paper deals with a phase of surveying that has received little attention in the past. With the possible exception of organizations charged with the execution of extensive surveys over large areas, engineers have fallen into the habit of using more or less rule-of-thumb methods of deciding what instrumental equipment and field procedure is to be used for a given survey. It is undoubtedly true that a vast amount of time and effort can be saved by properly drawn specifications regarding the necessary accuracy of the various steps in the making of a survey. It stands to reason that such specifications should be based on thorough studies of the effect of the various sources of error, considered singly and in combination, on the accuracy of the final results.

The writer believes, however, that specifications should be drawn for the several classes of surveys rather than for individual surveys. Except, perhaps, in the case of property surveys, it is not usually known beforehand what the shape of the traverse will be, nor is it usual to have any very detailed knowledge of conditions to be encountered along the route of a given traverse. This is especially true in certain parts of the country where, within the small space occupied by a farm of 100 acres, the character of the land may vary from almost perfectly flat bottom-land to the most rugged side-hill topography.

The concept that an error must be an unknown quantity in both sign and magnitude is open to discussion. The error due to phase in pointing on a signal is certainly systematic, but its sign is not necessarily unknown. The definition of a "Systematic Error" is likewise open to question, although it is true that it derives its name and character from the fact that its algebraic sign remains the same. The fact that this sign is always in doubt seems to be subject to debate. Take, for example, a cross-wind blowing against an unsupported tape. This certainly introduces a systematic error in the measurements, but the sign of the error is positively known since it always shortens the distance between the terminal marks of the tape and, thereby, causes measurements under these conditions to be too long.

That the effect of a systematic error cannot be eliminated by systematic field methods appears to be also a debatable subject. Suppose that a line is taped along a smooth concrete roadway, and a finely pointed pencil is used to mark the tape ends. It is conceivable that a tapeman might have a persistent tendency to mark to the right of the graduation at the end of the tape by a small amount. This tendency certainly would produce a systematic error.

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Now, suppose the measurement to be progressing toward the forward contact man's right, as he kneels by the tape. If he has a tendency to mark to the right he will be marking ahead, and his resulting measure will be too short. Suppose, then, that the measure is repeated in the backward direction, but that the observer stays on the same side of the tape. His tendency to mark to the right will then cause him to mark behind and his measure on the backward running will be too long. The mean of the two measures should then be free from his personal equation or systematic error in marking.

The first part of the definition of a "Cumulate" represents what the writer is in the habit of referring to as a correction. Any systematic error in surveying that can be eliminated by calculation takes on the character of a correction and does not affect the agreement of the final results. Any source of systematic error which can be, and is, eliminated by systematic field methods never appears in the final discrepancies at all. It would seem to be wise to retain the term, "correction", as referring to a systematic error for which an allowance is made by computation, and, since the other feature of the definition of a "cumulate" fails to enter the final results when proper field methods are used, it would seem that the necessity for the new term is slight at best.

"Mistakes" are the result, not only of confusion in the mind of the observer, but also of confusion in the mind of the recorder or from misunderstandings arising between them. If mistakes are of a size considerably larger than the errors of observation, they are detected because of their lack of conformity when measures are repeated but, in the case of small mistakes of a size equal to or only slightly larger than the errors of observation, they may be easily mixed up with legitimate errors and be treated accordingly.

Regarding temperature errors in tape measurements: If the temperature is assumed to remain constant, as suggested, the error is systematic to be sure, but if the assumed temperature is between the maximum and minimum actually encountered during the course of the work, the sign of the error will change, depending on whether this temperature is above or below the actual temperature at the time, and the resulting errors will not be strictly systematic throughout the whole traverse. It is noted that, in Table 1, Professor Rayner has computed values of E_t to the nearest 0.005 ft. per 100 ft. The writer suggests that this should be the nearest 0.001 ft., in order to be consistent with values of E_s in the same table.

To use the telescope reversed on the backsight and direct on the foresight at the odd numbered set-ups, and direct on the backsight and reversed on the foresight at the even numbered set-ups, will merely serve to place the error of adjustment of the line of collimation alternately plus and minus as the instrument progresses around the traverse. It seems to the writer that it would be better to let the error accumulate in a closed traverse and then eliminate it by distribution after the traverse is closed. Theoretically, this would give the correct angle at each station where the suggested procedure would make each angle in error by the same amount, but with the sign changing at each set-up along the traverse. In work in which considerable accuracy in angles is required, a reversal at each station is desirable; reading double deflections or using the transit as a direction instrument will accomplish the desired end.

In the comparison of the computed probable errors, under the heading "Errors in Chaining and in Angles Combined", the author states that the systematic errors were eliminated from the closed traverse, but could not be eliminated from the open traverse. This carries the idea, by inference at least, that systematic errors in an open traverse are more dangerous than in a closed one. It is true that the systematic errors which operate throughout the entire closed traverse do not appear in the final closure, but they are present, nevertheless. If a systematic error of 1 part in 1 000 is present in a closed traverse the accuracy of the survey is only 1 part in 1 000, even if luck is with the surveyor and he happens to get a perfect closure on his return to the initial station.

Under Example 6, Sub-division (*a*), the permissible error (1 ft.) is substituted as the probable error in a formula. To the writer this seems to be the misuse of a perfectly good formula. The total permissible error should not be assumed to be equal to the probable error.

W. H. Rayner,* Assoc. M. Am. Soc. C. E. (by letter).—In the discussion of this paper, exception is taken to the assumption that an error is a quantity of which both the magnitude and the sign are unknown; and also to the corresponding definitions of "Systematic Error" and "Cumulate".

It has seemed to the writer that there always has been an element of confusion in the discussion of the subject of errors because the term, "error", has included both the significant and the non-significant quantities which constitute the difference between any measured value and the corresponding true value. In the paper the significant and non-significant quantities were defined as "error" and "cumulate", respectively.

The analogy is not complete, but there is a similarity between the terms, "cumulate", as used in this discussion, and "tare", as used in weights. To weigh certain commodities a "gross" and a "tare" weight are taken, from which the "net" weight is obtained. The tare is sometimes found by weighing a container and sometimes it is compensated for, by a counterpoise. Thus, the tare weight may be found by a computation (that is, a correction), or it may be eliminated by a systematic procedure. The term, "error", as it has been commonly used, may be said to correspond to a "gross" measurement, including both the significant ("net" corresponding to "error") and the non-significant ("tare", corresponding to "cumulate") quantities.

If the proposed meaning of the term, "error", is accepted, namely, that it is that part of the difference between a true and a measured quantity which remains after the cumulate has been removed, then it is believed that the definition of the term "systematic error" as stated, is correct.

In the discussion it has been stated that,

> "That part of the definition relating to the sign does not seem to be correct or essential, because the error due to the wind effect of a suspended tape is always such as to make the measured length too large, as is also the error due to lack of alignment of a tape. Other examples of the same tenor could be cited."

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Mr. Rappleye also uses the same example and the additional one that "the error due to phase in pointing on a signal is certainly systematic, but its sign is not necessarily unknown."

In both these statements the sign of the "gross" error is known; but if so, would it not be logical to make an estimated or calculated correction? Thus, the effect of the wind on the tape may be estimated and the correction subtracted from the measured length. After this estimated correction has been applied (that is, after the cumulate has been removed) one cannot then say whether the remaining error is plus or minus, because the estimated correction may have been too large or too small. Similarly, after a correction for phase in sighting a signal has been applied, one cannot say what the sign of the remaining error will be. In passing, it may be stated that the distinction between the terms, "cumulate" and "correction", is that they have opposite signs. The cumulate is the quantity that is removed by applying the corresponding correction. However, probably little harm is done in using the terms interchangeably.

Mr. Rappleye further questions that part of the writer's definition of a systematic error which states that "its effect cannot be removed by systematic field methods". He uses the example of a forward chainman who has a tendency to mark to the right of the end of the tape, the effect of which is eliminated by the process used in measuring forward and back.

Here, again, he is using the term, "error", in its customary meaning and not according to that proposed. To the extent that the effect of measuring to the right is eliminated by a systematic field procedure it is non-significant and has been rendered a cumulate. In this instance the "tare" quantity has been obtained, not by a calculation but by a "counterpoise", that is, a systematic procedure. Surely it must be admitted that a final error remains, due to the effect under consideration. This remaining error, however small, is the significant or true error, and one cannot say what its sign or magnitude is.

The writer agrees that a "systematic error" need not always have the same sign. Of course, such conditions as temperature, direction of the wind and of the sun's rays, change from time to time; but any one of these conditions causes a systematic error only under the condition that its effect has the same sign for a considerable number of consecutive observations. Accordingly, the definition is improved by omitting the word "always", making it read "an error which has the same (unknown) sign, etc."

It may be added that the discussion of the definitions of the terms used is, after all, of little importance as compared with the major proposal with which the paper deals.

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Paper No. 1742

COMMENTS ON THE DESIGN OF SLUDGE DIGESTION TANKS*

By Richard H. Gould,† M. Am. Soc. C. E.

With Discussion by Messrs. Harrison P. Eddy, John R. Downes, Willem Rudolfs, Edmund B. Besselievre, Jerry Donohue, Kenneth Allen, and Richard H. Gould.

Synopsis

The digestion of sewage solids is brought about by certain fundamental processes that are common in Nature. These were utilized by Man for the treatment of sewage before the development of the science of bacteriology and before any proper appreciation was had of their nature. Metcalf‡ has given an excellent outline of the use of digestion in the cesspools of London and Paris early in the Nineteenth Century, of the application of the principles involved to small household appliances, and, in the latter part of the century, to the septic tank.

The septic tank has been largely superseded by the Imhoff tank, and in the past few years there has been a decided movement toward the digestion of sewage solids in separate tanks. These developments have been brought about in large measure by efforts to minimize the nuisances attendant on the treatment of sewage; to make the tank effluent more amenable to further treatment; and to reduce the costs of construction and operation.

The purpose of this paper is to indicate the manner in which the experiences of past years in the use of tank treatment and the recently developed scientific knowledge of the bio-chemical actions, are being applied to the design of sludge digestion tanks.

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† Chf. Engr., Ward's Island Sewage Treatment Works, New York, N. Y.

‡ "The Antecedents of the Septic Tank", Leonard Metcalf, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XLVI (1901), p. 456.

Experiences with Septic Tanks

The long use of septic tanks and cesspools has shown that the anaerobic digestion of sewage solids will reduce these solids very largely in bulk and will leave a residue of an inert, odorless character. This resulting sludge can be handled with relative ease and dried readily. Thus, the septic tank marks a distinct advance over the plain sedimentation tank and brings about a reduction in operating costs through easier sludge handling. Because of the difficulty of properly disposing of the sludge from plain sedimentation tanks the development of odors following septic treatment was probably not increased.

Certain evils, however, attended the use of the septic tank. The sewage became highly charged with odorous gases of decomposition, which were often liberated to the air during subsequent treatment. The effluent did not re-act so easily to aerobic treatment as fresh sewage effluents. During warm weather, at times of heavy gas formation in the tanks, the effluent was contaminated with suspended matter, often to an amount in excess of that in the raw sewage. The usual experience was that considerable odor was developed in the tank itself, both from the gases rising out of the liquid and from those produced in the mats of scum on top.

Experiences with Imhoff Tanks

The advantages of the digestion of sewage solids, as shown by the septic tank, were utilized to even better advantage in the Imhoff tank. While retaining the anaerobic decomposition of solids, this tank permitted the flowing sewage to retain its freshness by confining it to a special compartment separated from the decomposing sludge and by allowing the freshly deposited solids to pass to the sludge compartment by gravity through slots constructed so as to prevent the contamination of the flowing sewage by gas-lifted solids. Only the oldest and most thoroughly digested sludge is removed from good Imhoff tanks, and is probably somewhat better than that from septic tanks. The control of digestion is such that no odorous gases are liberated from the sludge compartment. A properly designed and operated Imhoff tank is still considered eminently satisfactory for the handling of sewage solids and the preliminary treatment of sewage.

Although there are many successful Imhoff tank installations, others to a considerable number in the aggregate have given disappointment. Perhaps in the majority of cases this failure can be traced to faulty operation, but often the original design has been at fault. Too often units for capacities that have proved satisfactory, have been applied to new designs without proper appreciation of the loadings, form of tanks, and climatic conditions under which the first results were obtained. Eddy,* in an exhaustive comparison of four Imhoff tanks, two of which gave satisfactory results, summarizes reasons for differences in behavior under twenty-five separate headings. The multiplicity of these considerations, many of which are entirely neglected by some and interpreted in accordance with individual bias by others, seems to explain adequately the reason why there are so many poor designs. Fortunately, later researches have tended to minimize some of the uncertainties formerly existing.

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* "Imhoff Tanks—Reasons for Differences in Behavior," by Harrison P. Eddy, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 465.

Granted adequate design, there are still reasons in some cases for a search for better appliances. The Imhoff tank with its two stories, one above the other, is a deep structure. The usual location for disposal plants is in low ground with a high ground-water level. The result is heavy construction costs for deep tanks. The fact that the sedimentation compartment and the sludge compartment of an Imhoff tank are combined in one unit makes it impossible to change the relation between the two. It is sometimes difficult to estimate in advance the exact flow to be treated and the sludge quantities to be cared for; indeed, these factors may change their relation after a number of years of operation. The tank, however, is fixed and cannot be altered to correspond. The temperature in the sludge compartment is essentially the same as that of the flowing sewage. In northern climates the cold winter sewage retards digestion greatly. Furthermore, it is not practical to heat the digestion compartment artificially, and gain the resulting benefits.

Early Example of Separate Digestion

The digestion of sludge in tanks separated from the flowing sewage has been successfully accomplished in England and in Germany. In America the first notable example on a large scale was at Baltimore, Md. There, following unfortunate results with Imhoff tanks, separate sludge digestion was developed. The early work did not have the advantages of previous large scale experience or of scientific research. Certain unanticipated results were obtained. At times, sludge that was expected to settle came largely to the surface; and there were other difficulties in handling the sludge, in lack of capacity, and in odor production. It was demonstrated, however, that separate digestion was possible and that its costs were reasonable. Under the conditions and arrangements provided, large unit capacities were found to be necessary.

Research on Digestion Problems

A most promising step toward better design of digestion tanks lies in the scientific research carried on within the past few years. The important work done at the New Jersey State Sewage Experiment Station at Rutgers College, New Brunswick, N. J., by Dr. Willem Rudolfs has been checked, in many respects, by parallel studies at Harvard University by Fair and Baity under the Rockefeller Foundation Fellowship. Other experimenters have been at work elsewhere in the United States, as well as in Germany and in Russia.

In carefully controlled laboratory experiments, it has been possible to separate the many variable factors that affect digestion and ascribe limiting results to each factor. It should be recognized that the course of digestion in the sludge compartment of an Imhoff tank and in a separate digestion tank is essentially the same, so that knowledge of the surrounding conditions can be applied with equal force to either type of structure. The form of the works may be different and this may alter environmental conditions, but the fundamental processes are the same in each case.

Nature of Digestion

Laboratory experiments have demonstrated that in the course of digestion the nature of the early products is acid, followed in the later stages by alkaline products. The most advantageous digestion occurs within a narrow range of acidity and this is best expressed in terms of the hydrogen-ion concentration, as pH-values. Should the pH-value become much less than 7.3, acid foaming may occur; and if much more, that of an alkaline nature may result. Odors result from either extreme condition. Thus, by controlling the pH-value within a narrow range the desired type of digestion—either liquefying or gasifying—may be secured.

Measurements of gas production have been made, and these, in conjunction with actual measurements from tanks in service, indicate that methane gas in worth-while quantities can be collected. As would be expected, the temperature has a large effect on the rate of digestion. Oxygen was found to be detrimental to anaerobic digestion, and to aid, perhaps, in odor production.

Practical Application

The essentials for optimum digestion are few in number and easy to apply. The question of reaction control in tanks of adequate design will normally take care of itself. It is only necessary to ensure that sufficient quantities of sludge in the later stages of digestion, when alkaline products are being given off, are in the tank, to counterbalance the acid production of freshly added solids. This is the criterion that should control tank loading. It is not the volume of tank space available, but the actual presence of a definite quantity of sludge that is important. Dr. Rudolfs believes that the addition each day of fresh solids to the extent of 2% of those already in the tank, both on the dry basis, is permissible without adversely affecting the optimum conditions for digestion, when the temperature is about 70° Fahr. The value of the artificial control of the reaction by the use of lime has been demonstrated in restoring tanks to proper operating conditions. This permits loading beyond that indicated.

The laboratory work is based on the quantity of destructible organic matter in the solids to be treated. The danger of using uniform per capita allowances for sludge digestion space has been frequently shown. Such a unit is convenient and has the value of time-honored use, but it should be defined in such a way as to correspond with sludge of the same volume and character as that deposited from sewage of normal strength under the usual settling tank conditions. On this basis laboratory results showed the digestion capacity required for a particular case to be about 0.8 cu. ft. per capita, provided the reaction and temperature conditions were controlled, whereas about 2.5 cu. ft. per capita would be necessary without such control. This is a ratio of about 1 to 3 in favor of controlled digestion.

Comparison of Laboratory and Field Results

Engineers acquainted with Imhoff tanks or digestion tanks, in operation under different conditions, will probably agree that the results shown by the laboratory tests through the control of digestion are not improbable. In comparing such tanks it should be borne in mind that the sludge capacity for the Imhoff tank is computed as that part below the slots, or even 18 in. below, whereas that of a sludge digestion tank includes the entire water volume. The space for floating sludge in an Imhoff tank may add but little to its digesting capacity; but there is also a space for liquid in most digestion tanks which must not be neglected in any computations for required capacities.

An example of a plant operating under unusually favorable conditions with low digestion capacities is the original installation at Dallas, Tex., designed by James H. Fuertes, M. Am. Soc. C. E. These Imhoff tanks, beginning in January, 1917, operated satisfactorily up to a loading beyond that for which they were designed. The digestion space below the slots was less than 0.5 cu. ft. per capita, as determined by a careful population check, modified somewhat by the volume of suspended solids actually being retained in the tanks. At this loading, however, well-digested sludge was obtained. It was inodorous and could be removed from the drying beds after three to four days of good weather. For a comparison with digestion tanks, the total space in the digestion compartments of these tanks (including that for floating sludge) was equivalent to about 0.9 cu. ft. per capita. It should be noted that the average seasonal temperatures were relatively high and that, in addition, a considerable volume of hot water came from artesian supplies. The range of temperature in the tanks is not known. The form of the sludge wells at Dallas is such that very nearly the entire capacity is available for digestion. Each well serves only one sedimentation chamber.

The loading at Dallas was much higher than is common in America, but it will be recalled that the early recommendations by Dr. Imhoff, for digestion capacities for domestic sewage from separate systems, were not greatly in excess of this quantity. These recommendations were based on the operation of similar plants in the Emscher District, in Germany. The writer has seen a number of those plants in successful operation.

A brief comparison of the per capita digestion capacity at Dallas with that at other more northerly locations is shown in Table 1. This confirms the conclusion that by suitable control it is possible to reduce the required digestion capacity to one-third that otherwise necessary.

Additional confirmation of the conclusions indicated in Table 1, is furnished by results secured at the Joint Disposal Works of Plainfield, N. J. In this case separate digestion tanks, constructed to handle sludge formerly treated in Imhoff tanks, are provided with floating covers, gas collection appliances, and means for artificial heating. John R. Downes, Superintendent and Designer, states that the new tanks have produced good sludge, without odors, under conditions of operation with a continuous loading such that there was only about 0.75 cu. ft. of digestion space per capita. The temperature range was from 65° to 70° Fahr. Formerly, unsatisfactory results were obtained with this same sewage in Imhoff tanks with a nominal digestion capacity below the slots of 1.49 cu. ft. per capita.* It is doubtful in this latter case whether all this space could be used to full advantage.

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* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 486.

TABLE 1.—COMPARISON OF CAPACITIES.

Plant.	Sludge capacity, in cubic feet per capita.	Ratio, to Dallas, Tex., as base.
Imhoff Tanks (Capacity to Slots):		
Dallas, Tex	0.5	1.0
Requirements, New York and New Jersey State Boards of Health	2.0	4.0
Fitchburg, Mass.*	1.88	3.76
Rochester, N. Y.*	2.49	5.0
North Shore Sanitary District, Illinois†	3.0	6.0
Digestion Tanks (Capacity to Water Surface):		
Dallas, Tex	0.9	1.0
Baltimore, Md.‡	3.57	3.97

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 486.

† *Engineering News-Record*, September 18, 1924.

‡ *Loc. cit.*, July 29, 1926.

CONSIDERATIONS AFFECTING DESIGN

How far the possibilities of digestion control may be utilized depends to a large extent on the intelligence with which the plant will be operated and the general policy pursued in the construction of the works. From a theoretical standpoint it is distinctly economical to maintain relatively high temperatures and to control the reaction of the contents. The saving in interest on the extra cost of larger tanks would pay many times over for the heating and lime necessary.

It has been demonstrated that methane gas in worth-while quantities can be collected from digestion tanks. The heating value of this gas is probably more than enough to maintain the desired temperatures. In one case where it is proposed to return surplus activated sludge to the digestion tanks it is estimated that sufficient power can be developed for a system of partial activation. More important than these possible economies are satisfactory operating conditions and results. It is good engineering to allow a certain margin in capacity to assure these ends. Unless the plant is to be operated with intelligence it will be difficult to realize all the possible advantages.

In arriving at the digestion capacity to be provided the question of sludge removal must be carefully considered. For operation at the maximum rating, the loading of fresh solids should be uniform. The removal of sludge throughout the year is a distinct advantage in the operation of the tank itself. This is rarely possible under the usual climatic conditions. If sludge cannot be removed at all times additional capacity must be provided in the tank, or elsewhere, to tide over these shortcomings. The improvement in methods of sludge handling and sludge drying is thus of great importance and has a direct bearing on tank design.

After the required digestion capacity is determined, a form of tank must be selected that will give this capacity in effective space. A flat-bottomed tank does not adversely affect digestion, but may contain dead pockets (as in the corners), that will gradually become filled with inert material and thus may not be considered as effective digestion space. The desirability of providing multiple outlets or mechanical means for maintaining the usefulness of the entire tank capacity can best be determined by a study of the relative costs involved. The admission of fresh solids to a tank should be controlled so as to distribute them fairly evenly throughout the active zone of the tank and to cause their thorough seeding with the older sludge. A concentration of fresh solids in isolated points is to be avoided.

The preponderance of evidence seems to be strongly in favor of providing covers for digestion tanks. One of the major reasons is to suppress odors. In an open tank a certain amount of oxygen comes in contact with the digesting material and this has been shown to retard digestion and tend, if anything, to increase the odors. If the reaction is not correct a dense scum may form which becomes partly dried on exposure to the air and is the source of objectional odors and flies.

A number of different types of covers have been used or proposed. A simple elevated cover somewhat above the surface prevents the spreading of odors to a certain extent. As usually operated, however, this form admits air to the tank when sludge is drawn, with consequent disadvantage to the digestion process, and possible formation of an explosive mixture of gas. This type is not particularly well adapted to the collection and utilization of the gas produced, as it may permit contamination with air, when sludge or liquid is withdrawn.

Suppressed covers, by keeping the scum submerged, prevent its drying and aid in its digestion. This form has the disadvantage that, in case the tank tends to foam, resulting conditions are aggravated, as the gas and foam must pass through a constricted outlet smaller than the gas vents of an Imhoff tank. The admission of fresh sludge causes an equivalent displacement of liquid which may contain an appreciable quantity of suspended matter. It has appeared desirable in some cases to provide a separate compartment to store this overflow.

A third type, perhaps the most flexible, is the floating cover, such as is installed at the Plainfield works. It rises and falls with the admission and withdrawal of sludge, thus excluding oxygen at all times and ensuring that the gas is uncontaminated. The cover can be operated so as to submerge such scum as is formed, or should foaming develop the vertical displacement of the cover provides space for its accommodation. A modified design has been proposed, much in the form of the ordinary gas-holder, providing a cover and also space for storing the gas produced. For practical utilization of the gas, some such device seems desirable to tide over inequalities in the rates of production and use.

Fresh solids admitted to a digestion tank undergo a reduction in volume by liquefaction and gasification. The digested sludge through its changed character is much more compact than fresh sludge and increasingly so the longer it is allowed to stay in the tank. The sludge withdrawn always has a lower water content than that added. This means that, in order to maintain an adequate volume of old sludge in a tank, liquid must be withdrawn from time to time. By careful selection of the point of withdrawal it is possible to obtain liquid with but little suspended matter, which may be returned to the plant influent without causing much trouble. The quantity of this liquid is a very small part of the total flow; if it is not so disposed of, it may be filtered on a sand bed.

A separate sludge well, in connection with digestion tanks, is often of distinct advantage. Surplus liquid can be drained to it from the tanks. The solids can be settled out or floated to the surface by the use of alum, the clear liquid pumped to the main influent of the plant, and the solids returned to the digestion tanks. Such a well also furnishes a convenient place for the adjustment of the reaction by circulating sludge from the tanks through the sludge well, adding the necessary lime at this point. When sludge drawn from the sedimentation tanks has a high water content, the sludge well may be used for its concentration. This will be particularly advantageous during cold weather, avoiding the necessity of heating excessive volumes of cold water.

The design of digestion tanks cannot be divorced from that of the other disposal works, nor from the degree of attention that the works will receive when built. The rate and character of the loading, the length of the season over which sludge may be drawn, and the temperatures that can be expected are all factors of vital importance. If it is probable that the plant will not receive careful attention, it may be advisable to add appreciably to the capacity. A nice balance must be struck between the benefits to be secured and the costs of construction and operation necessary to bring them about. A solution that is proper in one case, may be quite inadequate elsewhere.

Typical Designs for Digestion Tanks

The designs for digestion tanks shown in Figs. 1 to 6 illustrate the general trends. While a number of interesting developments are shown, no attempt has been made to indicate all types of tanks built or proposed.

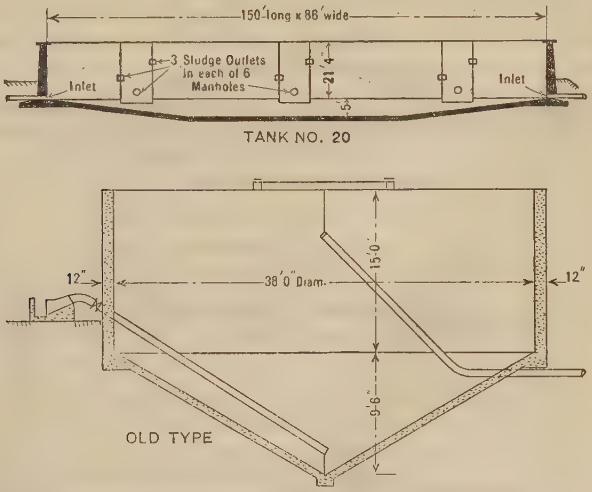

FIG. 1.—BALTIMORE, MD., TANKS.

The tanks at Baltimore, Md. (Fig. 1) are the largest and, at the same time, one of the first examples of separate digestion tanks in America. They use what might be called uncontrolled digestion. So far as is known, no

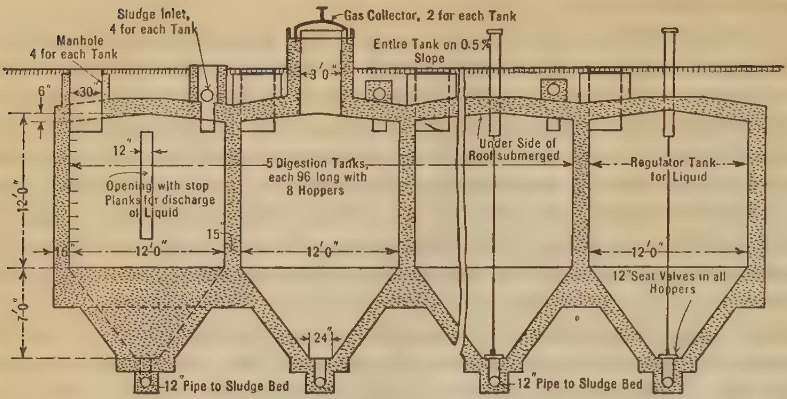
Sludge Inlet,
4 for each Tank
Manhole
4 for each Tank
Gas Collector, 2 for each Tank
Entire Tank on 0.5% Slope
3'0"
6"
30"
12'-0"
7'-0"
12"
Opening with stop Planks for discharge of Liquid
15"
16"
12'0"
5 Digestion Tanks, each 96 long with 8 Hoppers
12'0"
Under Side of Roof submerged
Regulator Tank for Liquid
12'0"
12"Seat Valves in all Hoppers
24"
12"Pipe to Sludge Bed
12"Pipe to Sludge Bed

FIG. 2.—BOONTON, N. J., TANKS.

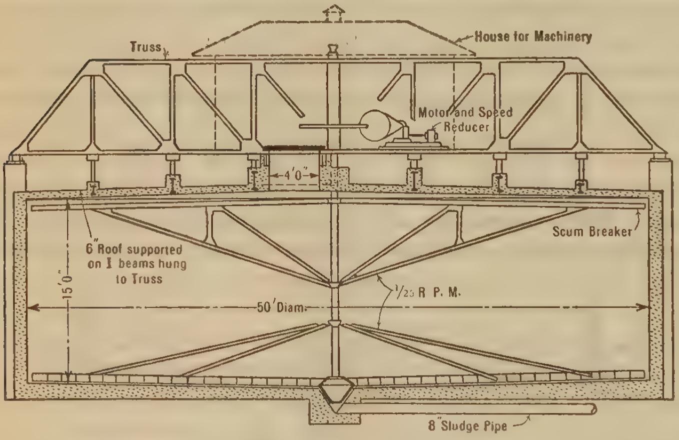
Truss
House for Machinery
Motor and Speed Reducer
4'0"
Scum Breaker
6"Roof supported on I beams hung to Truss
15'0"
1/25 R. P. M.
50'Diam.
8"Sludge Pipe

FIG. 3.—DORR DIGESTER.

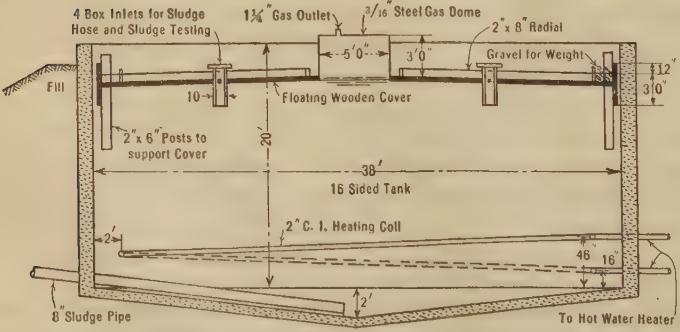
4 Box Inlets for Sludge Hose and Sludge Testing
1¼"Gas Outlet
3/16"Steel Gas Dome
2"x 8"Radial
5'0"
3'0"
Gravel for Weight
12"
3'0"
Fill
10"
Floating Wooden Cover
2"x 6"Posts to support Cover
20'
38'
16 Sided Tank
2"C. I. Heating Coil
2'
46"
16"
2'
8"Sludge Pipe
To Hot Water Heater

FIG. 4.—PLAINFIELD, N. J., TANK.

especial attempt has been made to limit temperatures and reactions, or to suppress surface scums and odors. However, they show conclusively that if sufficient capacity is provided, well-digested sludge can be secured. While some odor is unquestionably present, it is not serious when the size of the plant is considered.

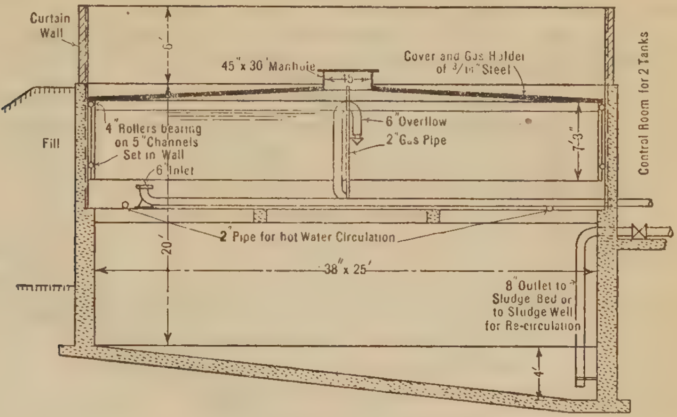
Curtain Wall
Fill
45" x 30' Manhole
45
Cover and Gas Holder of 3/16" Steel
6'
4" Rollers bearing on 5" Channels Set in Wall
6" Inlet
6" Overflow
2" Gas Pipe
7'-3"
Control Room for 2 Tanks
2" Pipe for hot Water Circulation
20'
38" x 25'
8" Outlet to Sludge Bed or to Sludge Well for Re-circulation
4'

FIG. 5.—LYNBROOK, N. Y., TANK.

The general features of the digestion tanks at Boonton, N. J., are shown in Fig. 2. This plant,* completed in 1924 by the municipal authorities of Jersey City, N. J., from designs by Clyde Potts, M. Am. Soc. C. E., for the protection of the water supply works at Boonton, has not yet (1927), been placed in service. The tanks are submerged under operation, so that no tough mat of scum will be formed. It is possible, however, to operate them with a lowered water level. Sludge is to be admitted at multiple inlets throughout their length; meanwhile there will be an equivalent displacement of liquid to a so-called regulation tank, held in reserve and having a lowered water level. Sludge is to be withdrawn through a system of steep-sided hopper bottoms. Provisions are made for collecting gas, which will be stored in a small holder near-by, and will be burned in a glass-covered sludge bed, or used to raise the temperature of the tanks, should it later be decided to install a heating system. The tanks are protected by an earth cover.

The tank shown in Fig. 3 is the so-called Dorr digester, of the gas-collector type. Others, built without covers and with simple elevated covers, are essentially similar in that they are plain cylindrical tanks with a flat bottom and provided with the Dorr digester mechanism. The purpose of this contrivance is to break up scum, to produce a mixing and circulation of the sludge, and to push it along the flat bottom to the sludge outlet. The Dorr Company has supplied the equipment for a number of such installations now in operation.

The digestion tanks at Plainfield, N. J. (Fig. 4), are thought to be the first in America to utilize the artificial heating of sludge by hot-water circulation and the patented floating cover. The cover, which is of simple wood construction, rises and falls with the admission or withdrawal of sludge or liquid and permits the collection of gas in the central dome. No trouble has occurred from scum, odors or foaming sludge. Sludge is admitted to the tanks through hose lines inserted in any one of four inlets in the cover, and is withdrawn through a single central pipe. The tanks were built at a very low cost.

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* *Public Works*, July, 1925.

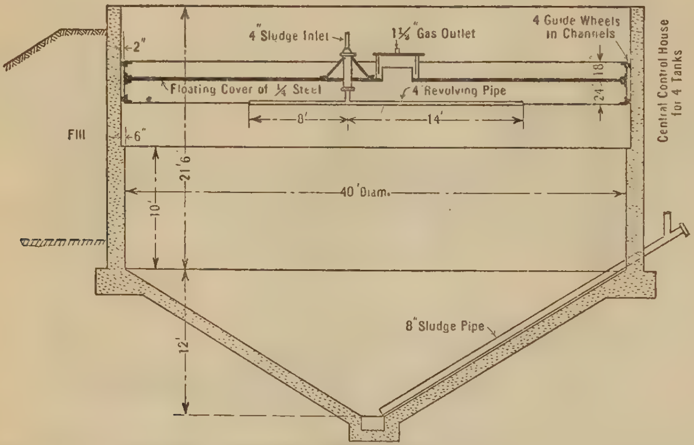

FIG. 6.—SPRINGFIELD, ILL., TANK.

The design for Lynbrook, N. Y., by Mr. Fuertes, is shown in Fig. 5. These tanks were intended both for primary solids and for surplus activated sludge. Provisions are made for heating the sludge, for circulating it, by pumping, for mixing and for reaction control. The unusual cover, of steel and in the form of a gas-holder, is to store the gas produced for utilization about the plant as required.

The design for Springfield, Ill., by the firm of Pearse, Greeley and Hansen (Fig. 6), is another example of an attempt to provide means for controlling digestion and collecting gases. A floating steel cover is proposed, with special means for introducing and distributing the sludge. The elevation of these tanks, as is true of all the examples cited, is not dependent on that of the flow line of the outfall sewer, so that appreciable economies in construction costs are possible by avoiding deep excavations in water-bearing soil.

FUTURE DEVELOPMENT

With the development of more accurate knowledge of the nature of anaerobic digestion, this process seems capable of being used with more intelligence than in the past. It furnishes an economical method for the inoffensive disposal of organic wastes and will undoubtedly have broad application. Information developed in the last few years has had a marked effect on the trend of design of sludge digestion tanks. There is, however, much to be learned as to the absolute capacities required under different conditions. To this end engineers should continue the generous publication of accurate operating data.

DISCUSSION

Harrison P. Eddy,* M. Am. Soc. C. E.—The changes which have been taking place in the art of sewage disposal since about 1918, are somewhat radical. For many years, the primary effort was to produce an effluent which would satisfy local conditions, and the problem of sludge disposal, while recognized as being troublesome, was subordinated in a measure. The tendency, to-day, is clearly toward more complicated treatment plants and the division of the processes into a number of stages.

Formerly, little attention was paid to the matter of odors produced at treatment plants. It was common practice to discharge raw sludge into lagoons or on to open drying beds, often resulting in the dissemination of offensive odors. At present, determined effort is made to prevent the escape of odors in so far as possible. Sludge digestion is now (1927), commonly practiced to accomplish this.

It has long been recognized that temperature has a material effect on decomposition, but sufficient attention has not been given to this phase of the digestion process. It is well known that, in the colder climates, streams which are so polluted as to create offensive odors in summer are relatively inoffensive in winter.

It has not always been recognized that the storage of sewage solids on the stream bed in winter is accumulating trouble for the following summer. In the colder climates of this country such solids are subjected to a refrigerating process for perhaps six months of the year. The temperature of the river water is so low that it prevents the solids from changing materially, but when summer comes, with an increase of 65° or more, in water temperature, decomposition of the accumulated solids creates a load on the stream which is greater than the current load.

The digestion of sludge in unheated tanks takes place under similar conditions, but by controlling the temperature, as Mr. Gould has indicated, active digestion can be maintained continuously and it is possible materially to reduce the storage capacity required. Heat may be provided by burning the gas evolved from the digesting sludge; but this introduces additional complications because of variations in the rate of gas production, the need for regulation of pressure, and the necessity for adequate precautions against accidental explosion.

Atmospheric oxygen, mingling with the gas in a tank, may form an explosive mixture. Such mixtures, containing between 5.5 and 12.8% methane, are explosive, and may be produced by many conditions, such as those following the partial removal of sludge from a tank.

There is difference of opinion as to the mechanical equipment which should be provided for separate sludge-digestion tanks. It is practically impossible to introduce the sludge uniformly throughout the tank. This difficulty may be overcome in a measure by artificial mixing. A mechanical stirring device may be placed at the bottom of the tank where solids are supposed to accumulate, but the solids may accumulate in the upper part of the tank, in which case stirring equipment would be more effective at the top. Experience has also shown that there may be an accumulation of solids in the middle of the tank, with comparatively clear water above and below. These conditions are probably due in large measure to the kind of biological action taking place, although temperature and other physical conditions undoubtedly are important factors.

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* Cons. Engr. (Metcalf & Eddy), Boston, Mass.

Some biological processes are carried out most advantageously at certain rather definite points of acidity or alkalinity. It is logical that there should be an optimum reaction for the digestion of sewage sludge. The reaction may be regulated by the addition of chemicals. Having established favorable digestion, it is important to keep a quantity of sludge in the tank sufficient for seeding the freshly deposited sludge.

In developing the art of sewage treatment, in an effort to prevent offensive odors, to secure a high degree of purification, and to handle the sludge under advantageous conditions, engineers are introducing processes and equipment which add operating complications. These also add to the cost of design, construction, and operation.

It is difficult to secure funds for proper operation, but it is more difficult to secure operators who have the technical knowledge and skill required to operate such plants successfully. This country has advanced from the simple, "fool-proof" sewage treatment plant of the Nineties, of which very little was required except good luck, to the highly complicated mechanical plants, like that at Milwaukee, Wis., and like some of the separate sludge digestion plants. The difficulties ahead in the field of operating these plants constitute one of the most serious problems confronting the engineer practicing in this field.

John R. Downes,* Esq. (by letter).—In the writer's opinion, the work of Mr. Gould stands out as of the greatest importance to the art of sewage disposal in that it sums up so accurately the present-day knowledge and information on sludge digestion.

Those who have been intimately connected with scientific investigations have realized for some time the necessity of having the *status quo* of developments presented from the engineering standpoint. Mr. Gould has presented the matter so ably that any analytical discussion would simply tend to befog that which he has made extremely clear.

He suggests that inert material might settle out in digestion tanks to the extent of decreasing the available digestion space. In reply it may be stated that in tanks of the design shown by Mr. Gould, in Fig. 4, which are practically flat-bottom tanks, only from 2 to 4 in. of gritty, inert material has settled out during a period of two years, notwithstanding the fact that a grit chamber installed at the end of the two-year period removed about 6 cu. ft. of grit daily from the incoming sewage. In the settling tanks and flow channels the grit tends to separate from the sewage and from the thin watery green sludge, but, in the digestion tank, it becomes rather intimately incorporated in the more viscous, ripe sludge and is carried out with the latter. This specific information was not available at the time the paper was prepared.

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* Supervising Engr., The Plainfield and Dunellen, N. J., Sewage Disposal Works, Bound Brook, N. J.

WILLEM RUDOLFS,* M. AM. SOC. C. E. (by letter).—This admirable paper is one of the best attempts to bring together and apply the findings in this field during the last few years. In this case a discussion from the standpoint of an investigator must confine itself to supporting evidence and possibly to the emphasis of some minor details of practical interest to the designing engineer.

The course of digestion of fresh sewage solids is essentially the same in septic tanks, two-story tanks, and separate sludge digestion tanks. However, the difference in the construction of these tanks indicating their use is of great importance in connection with the digestion capacities and the manner of operation. In septic tanks and two-story or Imhoff tanks, two distinctly separate processes are combined: (1) Sedimentation, which is a purely mechanical process; and (2) digestion, which is a purely biological process. Although there is no relation between the two processes, in a septic tank both must take place within the same tank, and in the case of an Imhoff tank there is a more or less intimate connection between the upper and lower compartments. In an Imhoff tank, sewage solids settle and fall through the slots and are supposed to remain there until they are decomposed. This is only partly true, since the solid particles (except at the bottom of the tank when very old sludge is present) are in constant motion, either up or down, depending on whether or not the particles carry entrained gas. Part of this material as well as a part of the liquefied material is displaced by the incoming fresh material. The displaced materials leave the tank with the effluent. From the standpoint of best possible results the processes of sedimentation and digestion, unlike in principle, cannot be combined because, in order to obtain the best possible sedimentation, the suspended solids and partly liquefied material should be prevented from rising and contaminating the effluent.

With separate sludge digestion tanks the replacement discharge does not take place; or, in other words, practically the whole volume of material added must be withdrawn. Studies on operating tanks that treat fresh solids have shown that only from 4 to 8% of the volume is reduced, due to biological activities and about the same percentage when activated sludge is digested. Obviously, when thin fresh solids are kept in a tank without further concentration the digestion capacity would be much greater than in a two-story tank. Fortunately, fresh solids that are added to a tank separate quickly from the water. This takes place in separate sludge digestion tanks as well as in Imhoff tanks, and thickened sludge in separate sludge tanks can be obtained by decantation, whereas in Imhoff tanks the decantation process takes place automatically, although it cannot be controlled. With control, the operator is able to prevent contamination of the effluent from the settling units and thereby prevent possible nuisances due to odors and difficulties in overloading of sprinkling filters.

Little information is available on the condensation of sludge from domestic sewage. Decantation of the comparatively small quantities of supernatent liquid is rather easy, but the question arises at once as to how far such displacement of liquor may be carried. Exhaustive laboratory experiments have shown that sludge concentration can be practiced to a high degree without detrimental effect to digestion activities.

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* Chf., Dept. of Sewage Research, State Agri. Experiment Station; Prof. and Head of Depts., Water Supplies and Sewage Disposal, Rutgers Univ., New Brunswick, N. J.

The longer the digestion schedule, the larger is the digestion capacity required. Assuming good operation, the effective digestion capacity of a separate sludge digestion tank can always be the same or less than for an Imhoff tank. As soon as control is practiced the digestion capacity of a separate sludge tank is considerably less. This is especially true in cases in which heat is applied. Heating of a separate sludge tank is a comparatively simple matter, whereas experiments conducted by Dr. Imhoff to heat the digestion compartment of an Imhoff tank failed because the temperature could be raised only from 2 to 3° Fahr. without excessive heat supply. Moreover, too great quantities of hot water introduced would tend to produce a thin sludge.

In the past, the designer did not have available a number of fundamental facts. Several factors influencing digestion, such as seeding, temperature, and reaction, have been investigated and are now largely available to the engineer. A few curves are submitted herewith to emphasize and illustrate the more important single factors.

The most important single factor known to the writer at present is the inoculation or "seeding" of raw sewage sludge with sludge that has undergone decomposition. Such sludge has a well-balanced group of organisms, and their enzymes, responsible for rapid decomposition, contain the necessary chemical constituents to counteract the effects of intermediate decomposition products formed during the early stages of digestion (Fig. 7). Although the total quantity of gas produced per given quantity of organic matter is not influenced by seeding, the material not inoculated requires about six times longer for digestion than the properly inoculated sludge at the same constant temperature of 70° Fahr.

The effect of temperature is second in importance (Fig. 8). Measured by the digestion time required, unseeded fresh solids at 50° Fahr. require six times longer than material digested at 82° Fahr., which is considered the approximate optimum. Only at very much higher temperatures could it be possible to reduce the digestion time further. With properly seeded mixtures the digestion time at 70° Fahr. is nearly five times shorter than at 50° Fahr. Although the information available is comparatively scarce, the average yearly temperature taken at a number of sewage disposal plants varies between 52 and 58° Fahr. It is evident, therefore, that sludge heating is of great importance in a northern climate, although it should not be forgotten that the average yearly temperature of the material in the sludge digestion tanks at Baltimore, Md., is only 52.5° Fahr. This brings up the important question of proper insulation of digestion tanks. If tanks are built above ground there should be at least an earth fill around them, although it might be better if some one would devote time to insulate them properly. Insulation is not only important when tanks are built above ground, but also when tank contents are influenced by ground-water.

The relation between changes in the reaction of the medium and the gross activities of the micro-organisms is shown in Fig. 9. For convenience, the accumulated gas production from a mixture of sewage solids, with the reaction controlled by hydrated lime, is compared with gas production of a mixture digesting under "natural" conditions. It should be kept in mind that this cannot always be done, because under certain circumstances liquefication might be too great. Although lime retards initial gas production, the amounts produced after 60 days, with the reaction controlled by the addition of lime, is more than two and one-half times the quantity produced from the unlimed material.

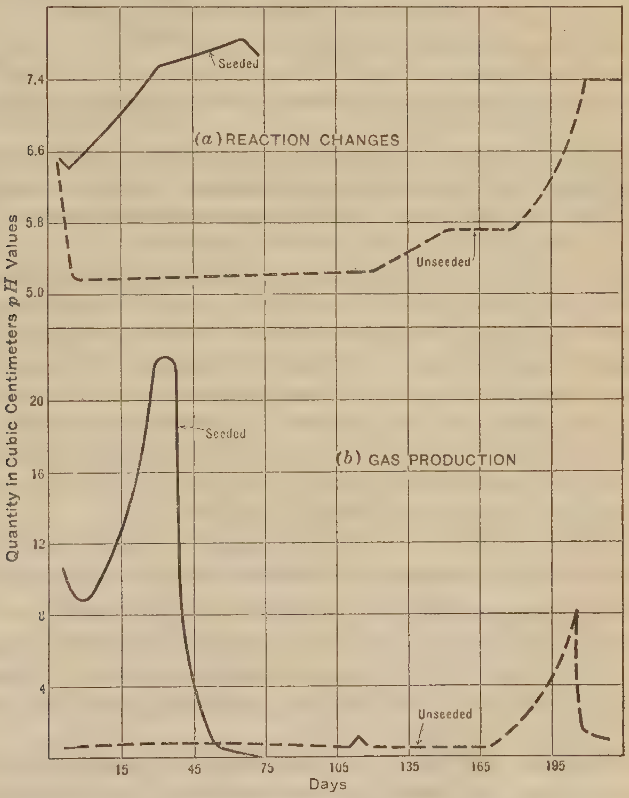

FIG. 7.—EFFECT OF SEEDING ON THE RATE OF SLUDGE DIGESTION.

As pointed out by Mr. Gould the gases produced by decomposition of sewage sludge can be utilized for heating the sludge, blowing air, running pumps, etc. The tendency is to heat the tanks by means of closed circulating hot-water systems with coils running around the tanks. It is obvious that such coils should be continuous. The writer knows of only one case in which the heating coils are placed vertically instead of horizontally, namely, in the new tanks built for the digestion of screenings at Milwaukee, Wis. Offhand it would seem that vertical coils are better from the standpoint of water circulation and possible caking. Experiments made at the New Jersey Sewage Experiment Station have shown that water heated to 140° Fahr. caused no caking.

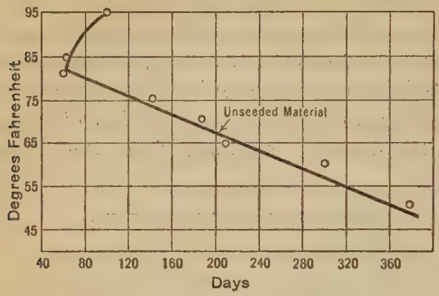

FIG. 8.—EFFECT OF TEMPERATURE ON RATE OF SLUDGE DIGESTION.

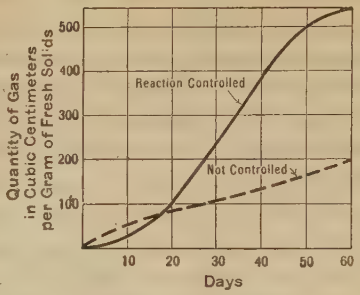

FIG. 9.—EFFECT OF REACTION ON RATE OF DIGESTION.

Since nearly the same volume added to a separate digestion tank must be withdrawn, means for withdrawal of supernatent liquid are important. An overflow at the top might carry with it some suspended matter. Additional "take-offs" at different levels would facilitate operation.

In addition to what Mr. Gould states about permissible loading of tanks based on sludge digestion capacity per capita, it should be kept in mind that with improved settling of suspended solids the volume of sludge to be handled by the digestion tank increases. Naturally, when more solids are retained in a tank the volume of sewage to be handled by a following oxidation device (like sprinkling filters) can be increased. Moreover, when solids are settled without contamination of the tank effluent by the partly decomposed material emerging from an underlying digestion unit, the odors emanating from such an oxidation device are at once decreased materially.

In conclusion, the writer would like to underscore Mr. Gould's plea, that engineers continue the generous publication of accurate operating data.

EDMUND B. BESSELIEVRE,* M. AM. SOC. C. E. (by letter).—No phase of sewage treatment is more timely than digestion of sewage sludge in separate tanks. At present (1929), unquestionably, more study is being given to the digestion of the solids obtained from settled raw municipal sewage than to any other phase of sewage treatment.

Within the past few years a great amount of experimental work, both laboratory and semi-full scale, has been carried on with the view to determining the important factors in the digestion of sewage solids, for use in designing the necessary structures. This work is of inestimable value; it has enabled engineers to appreciate factors that have been neglected in the past, and to make obsolete plants successful.

On the other hand, abstract theories derived from experimental work, cannot produce results on a large scale, unless all the technical and economic factors are carefully considered before the actual design of the digestion tanks.

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* San. Engr. and Mgr., Eastern Territory, San. Eng. Div., The Dorr Co., Inc., New York, N. Y.

The primary object of digestion tanks is to produce—in a short period of time, without odor, unsightliness, or other nuisance—an inert, inoffensive sludge which may be dried in the open air or otherwise disposed of without offense. This sludge should be of small volume, low in moisture, and free from odor when wet or dry.

In small plants these results should be obtained with the minimum of manual attention and in a minimum area. In large plants, area is perhaps the greatest consideration because adding one or two men may be cheaper than greatly increasing the area.

Observation made in the period since 1925 clearly demonstrates that such results may be obtained. When the digestion of the organic solids is complete, the resultant sludge may represent only 40% of the original dry weight. This lessened volume of wet sludge to be discharged on the drying beds in turn means that a smaller area of beds is required. With the modern plan of covering beds with glass, continuous use of all the drying area may be obtained regardless of weather, further reducing the area required. In small plants, where land is cheap, glass-covered beds at about $3 per sq. ft. may not be more economical than open beds, but this would seldom hold true in a larger plant.

The main considerations to be given careful thought in designing sludge digestion tanks are:

1.—Volume.
2.—Shape and number.
3.—Simplicity of construction.
4.—Operating devices for:
    (*a*) Controlling digestion;
    (*b*) Observing the progress of digestion;
    (*c*) Controlling the temperature;
    (*d*) Collecting and utilizing the products of digestion;
    (*e*) Removing digested sludge;
    (*f*) Handling the overflow of displaced liquor;
    (*g*) Admixture of new and old sludges;
    (*h*) Handling and overcoming scum troubles; and
    (*i*) Moving sludge from one part of a tank to another.

Volume.—In the older plants it was impossible to achieve satisfactory results without providing for much sludge storage. This, in general, has been due to the method of operation and not to any unusual characteristic of the sewage or its organic solid content.

As Mr. Gould has shown, the first step in the digestion cycle is an acid fermentation, which is followed to completion later by an alkaline series. Modern research has clearly shown that if a tank is filled with sludge undergoing acid decomposition, the entire tank will be acid. Unless this is overcome artificially, sufficient time must be allowed for the natural reaction to take place. This means large volumes, large tanks, and large area consumption. Dr. Rudolfs and others have shown, however, that if a fresh sludge is added daily to a tank filled with a mass already alkaline, the acidity created by the new raw sludge will not upset the reaction of the tank, and alkaline digestion will continue. This daily proportion has been roughly placed at from 2 to 3% of the total. Sludge that is not more than 24 hours old, preferably 8 to 16 hours, is defined as fresh, raw sludge. To obtain these results is not difficult and requires little technical skill. In plants that require only the part time of one ordinary laborer per day, successful sludge digestion has been accomplished over periods of years. Naturally warm climates are more favorable to continued digestion, but records show equally satisfactory results in cold northern sections.

Mr. Gould discusses the modern method of expressing the environment for successful digestion—the hydrogen-ion concentration, expressed as pH-values. Apparently, a misunderstanding is current. Several investigators have found that the pH-range for best results varied from 7.2 to 7.6, others from 7.4 to 7.6, or even more. Some operators have taken this to mean that it is necessary to maintain a pH-value of at least 7.2 to 7.4. This is not so. If 7.2 to 7.4 is found to be the best range for some plant, no one should infer or claim that it should be applied arbitrarily to every case. Intensities in pH-values of 7.1 and even in those as low as 6.8 have produced good results. Unless it can be proved that a certain pH-value is necessary to produce good digestion, no extra money should be spent on re-agents to procure a given degree of alkalinity. Only experimental work can determine the most economical limit.

TABLE 2.—RANGE OF pH-VALUES MAINTAINED AT VARIOUS SEPARATE SLUDGE DIGESTION PLANTS.

Location.	pH-VALUE.	
	Liquid.	Sludge.
Hartford, Wis.	6.8 to 7.0	6.7 to 8.1
Kiel, Wis.	....	7.1 to 7.4
Sheboygan, Wis.	....	7.4
Antigo, Wis.	7.2	7.4 to 7.6
Ridgewood, N. J.	....	7.0 to 7.4
Storm Lake, Iowa	....	7.2 to 7.6
Sioux Falls, S. Dak.	....	7.3

The fluctuation in necessary pH-values in different plants is shown in Table 2. These plants vary in size, characteristics of sludge, locality, operation, etc.

The volume of sludge digestion tanks is a matter of extreme importance. The customary unit of "cubic feet per capita" has led to misunderstanding, because it has not always been clear whether it was based on present, maximum, future, or some intermediate population.

Authorities now agree that a better way is to record the effective capacity on the basis of the solids received in the tank, that is, the cubic content of the digestion tank from the bottom to a point 1 ft. below the overflow. This point is taken to prevent the discharge of sludge through the over-flow pipe.

As temperature is important for procuring year-round digestion, a digestion tank for a warm climate would require less volume than one for a cold climate. If the warm temperature favors uninterrupted digestion throughout the year, aside from all other considerations, this would warrant the use of a small volume per capita. Conversely, in a colder climate for months digestion is inhibited, and the tank functions as a storage tank only; therefore, sufficient volume must be provided to accommodate the daily sludge contribution for the dead period. No arbitrary scale of the proper volumes per capita can be stated, as other elements besides temperature influence the speed of digestion. The maintenance of a favorable alkaline environment, the characteristics of the sludge itself, and the character of the solids, all tend to influence the digestion time and the volume required. Certain industrial wastes contribute such a large extra load of solids that generate acid products as to require from three to four times as much volume per capita as normal sewage sludge.

Tanks without artificial temperature regulation, and those in which the acid step is not controlled by proper additions of sludge or the use of corrective agents, have a lag in digestion because, during the period of acid fermentation, there is no digestion. This must be provided for in extra volume.

Knowing the optimum temperature and the correct proportioning of raw and digested sludge, an engineer can design tanks that will produce well-digested sludge throughout the year with a minimum of capacity.

Design data of fifty-one plants constructed in all parts of the United States since 1923 show that the average per capita volume in the digestion tanks is 2.87 cu. ft. The range is from 1.15 to 12.0 cu. ft., the latter being for a plant having a large proportion of industrial wastes.

Shape.—Assuming that control of temperature and reaction is to be provided, the next consideration is the shape of the tanks.

Construction cost enters into this largely. Simplification by using straight walls, omitting baffles and intricate sections, decreases the initial cost. The simplest form is a vertical cylinder, with no internal walls or baffles and a relatively flat bottom. This simplifies form work and reduces the amount of reinforcing steel, at the same time producing the strongest possible wall. It assures speed of construction and avoids the occurrence of pockets to trap gas.

Practically all the tanks built in the United States and Canada since 1924 have been vertical cylinders. Some have flat bottoms, cleaned mechanically; others have cone bottoms, depending on hydrostatic evacuation of the sludge. Some have flat roofs carried from the tank walls; the writer agrees with Mr. Gould that domed roofs are not only difficult and uneconomical to build, but introduce an element of danger in allowing the accumulation of large volumes of gas which may become extremely explosive if air is admitted to the tank. In modern tanks, with the liquid level maintained at the bottom surface of the tank cover, the only available gas collection space is the small dome, and the danger is negligible.

The number of tanks required is a matter of argument. A study of fifty-one plants constructed since 1923 shows that in all except the largest the number is usually two. As tanks up to 85 ft. in diameter have been in operation for some time, and those of larger diameter may prove equally practical, there is no mechanical reason for providing a large number. Doubtless, economy is best served by fewer and larger tanks. As digestion tanks are provided for the sole purpose of allowing organic solids to undergo a natural process of decomposition, a larger capacity than is required at the start is no detriment. Therefore, fluctuating population is not a good argument for a large number of small tanks.

As no definite advantage has been shown in progressive digestion, that is, transferring sludge from one tank to another, over controlled digestion in one tank, no economic purpose can be served by more than two or three units. Furthermore, in heating one or two large units, the heat loss due to the addition of daily sludge will be less than in smaller units; therefore, it should be possible to maintain more equal temperature. Less heat also will radiate, as the aggregate wall area of the larger units will be less. Furthermore, in places where mechanical units are used for distributing and collecting sludge and releasing gas, a few large units will cost less initially and less to operate than a larger number of small units.

Operating Devices.—The next considerations are the means for controlling digestion and temperature, for collecting and utilizing products of digestion, for eliminating scum, for mixing new and old sludge, for removing displaced liquor, etc.

As one of the products of digestion is gas, which when disseminated in quantities in the atmosphere is productive of obnoxious odors, the modern idea, as expressed by Mr. Gould, is to cover the tanks. This is true regardless of locality. Tanks operating, for instance, in the South, would not normally require covers, because there is no need for heating the tanks, and, therefore, for collecting the gas to utilize its heat value; but from the standpoint of control of odor, an uncovered tank is never a certainty, therefore, even in warm climates, covering is frequently advisable, especially where plants are close to residences. The gas may be burned merely to provide against odor, or it may be used as a source of heat or power as required. In any section where a period of cold weather may reduce the tank temperature to the danger point, heated and covered tanks are proper.

Research and observation show definitely that a separate sludge digestion plant with modern digestion units, controlled temperature, and reaction, treating an average municipal sewage will produce sufficient gas to maintain the optimum digestion temperature throughout the year, and, in most cases, will provide heat for the plant buildings and frequently for the glass-covered sludge beds as well. This economy is provided at no cost.

At Antigo, Wis., the cost of electrical current to operate all units of the plant for the eighteen months beginning July 1, 1927, averaged $30 per month. The total production of gas was 2 300 000 cu. ft. Assuming that this gas had been sold at a rate of 30 cents per 1 000 cu. ft. (a low rate for a high-grade gas), the total revenue would have been $690. On this basis the gas produced would have offset the entire cost of electric current for the period. As a matter of fact, the gas was used to heat the digester and the plant buildings; if the required gas had been purchased locally, it would have cost $215.25 per month. If soft coal had been used, it would have cost more than $1 000. With an ordinary circular tank, the apparatus is moderate in cost and simple to operate and control.

Mr. Gould refers to the formation of scum in digestion tanks. Even with reaction control scum may form, but, as he states, it is more likely to occur with incorrect reactions. Scum that forms in a tank with gas collection domes in the cover prevents the free and even discharge of the gas and may interfere with the uniformity of flow to the heating system, requiring a holder of considerable volume to equalize the flow. A tank equipped with a revolving mechanism to break down the scum continually will provide an even flow of gas and obviate the gas-holder, except in very large plants where it is desired to store the surplus gas during periods of warm weather. A mechanism of this type is shown in Fig. 3.

A digestion tank, particularly a closed one, should be kept full of sludge and liquid at all times. Withdrawal of sludge to the drying beds will cause the level of the liquid in the tank to lower unless the period of withdrawal is timed to correspond with the period of sludge addition, and this is a simple accomplishment. If the over-flow pipe is trapped and baffled properly, there will be no chance for air to enter the tank during withdrawal periods, and little if any chance for solids to pass out of the tank.

The liquid displaced from the digestion tank is naturally equal in volume to the amount of sludge put in the tank, except during the period when sludge is being withdrawn and admitted at the same time. A large number of plants return this displaced liquid to the inlet side of the sedimentation units. Most of the settleable solids that might pass out of the digestion tank with this liquid are re-settled and returned to the digester, and the volume of this liquid displaced in any one day, as compared to the total volume of sewage flowing through the sedimentation unit during the same day, is so small that, in general, this return of displaced liquid would not upset the operation of the sedimentation tank. However, several State Departments of Health have issued rules requiring the treatment of this displaced liquid separately. This usually entails only the construction of a small sand filter.

Digestion tanks may be heated in several ways. The simplest and cheapest method is to circulate the gas-heated water through the tank by a coil of pipe on the walls. The first form used was similar to the familiar factory heating coil with several pipes running to manifolds at each end. This was found to give a little difficulty in heat loss and ineffective heating of all parts of the coil, so that the favored type is a spiral running completely around the inside face, starting about 2 ft. from the bottom of the tank and having four to six complete turns. As the hot water must go in one end and out of the other, there can be no tendency to clogging or sluggishness of flow.

Observations have shown that the initial temperature of the hot water should be sufficient to transfer approximately 20° Fahr. to the sludge to maintain the optimum range of 75 to 80° Fahr. There is, however, a danger point to be avoided here. If the initial temperature is too high, the sludge may dry and cake too rapidly around the sludge pipes. This will make them ineffective as heating units, and cause a continuing increase in the initial temperature of the water. By experience even in the extreme northern section of the country (Antigo, Wis.), an initial temperature of 120° Fahr. will allow continued satisfactory operation and no caking will result.

Other means of applying heat to the sludge have been suggested, such as circulating hot water through it, depending on the convection currents to warm the entire mass. The writer believes that this method will entail greater heat losses than the pipe-coil method, as the hot water will naturally rise to the surface and pass from the tank with the over-flow; and that in passing upward it will not diffuse through the entire mass, but will flow upward in streams through the sludge, heating only the portion directly adjacent to it in its travel.

When the sludge is completely digested, it should be removed to the drying beds, thus freeing space for new sludge. If hydrostatic evacuation is relied upon solely to move the sludge, this evacuation will not be complete and certain volumes will remain upon the walls and bottoms, caking and eventually reducing the effective digestion space. True, this may be prevented by sprays of water or air operated at the time sludge is withdrawn, or by hand-squeegeeing the slopes; but there will be no guaranty of complete evacuation. The addition of water increases the moisture of the sludge, the driving period, and the volume of over-flow from the top of the tank. Air is apt to cause flotation of solids, forming surface scum. Hand-squeegeeing introduces an element of cost and is not entirely satisfactory. Any of these three methods requires eventual dewatering of the tank and careful cleaning of the surfaces. By means of a mechanism (see Fig. 3), complete evacuation of the sludge may be obtained.

If the sludge is allowed to compact in the bottom of the tank, without any attempt at stirring to procure an admixture between the old, partly digested sludge and the new raw sludge, digestion will be stratified. By providing a means for gently stirring the mass, keeping it porous, and allowing the continuous upflow of the gas in small bubbles, the formation of thick surface scums, which is so common in many tanks, is prevented.

One of the recognized features of modern plant operation is the control of alkalinity in the digestion tanks. This usually consists of adding sufficient lime to the sludge to overcome the acidity. It is simpler to add the small quantity of lime required directly to the daily volume of sludge than to the entire tank volume. Hence, the tank which permits this will be an economical unit in operation. In several cases in the writer's experience with plants of this type, when no attempt at reaction control was made at the start, it has been found that after a period the entire tank contents were acid. It was then necessary to add sufficient lime to the tank to bring the entire mass to the alkaline sludge and after that to add sufficient lime each day to the new sludge admitted to maintain this alkalinity until tests showed that the tank was entirely in alkaline balance and the daily addition of new sludge would not upset it. This was a very simple operation with the mechanical type of digester. The lime for correcting the tank mass was simply dumped in at the top of the tank and the mechanism worked it down into the sludge; the lime required for the correction of the daily volume was added in the open flume where the sludge was received from the pump at the top of the tank. In one case (Lakeland, Fla.), this entire process occupied a period of about six weeks, and since then the tank has operated most satisfactorily, without odors and without the addition of any lime for more than two years.

Distribution of sludge to all parts of the area of the tank is important to procure the prompt seeding of the raw sludge and to insure complete digestion. The mechanical type of digester provides this, as the revolving mechanism tends to distribute the sludge thoroughly to all points in the tank.

In the writer's opinion, the entire question of design is most important, and engineers in general cannot give too much time to the consideration of every factor in the endeavor to produce plants which will function in a satisfactory manner. The trend of thought as regards sewage treatment works is rapidly changing for the better, and municipalities now are looking upon them as important utilities and are willing to spend the sums necessary for proper construction. The modern sewage treatment plant using the proved means for digesting sludge may be constructed with safety, in sections close to residential districts or near main highways without being considered eyesores or sources of nuisance.

JERRY DONOHUE,* M. AM. SOC. C. E. (by letter).—The advance in the art of sewage disposal actuated by a public demand for better treatment plants has been most pronounced in the period since 1900 and indications point to still further perfection in the science in the future. Problems of sewage disposal are now public questions which demand study and analysis not only by municipal officials but by the taxpayers who are called upon to pay the bill and who are demanding better treatment plants. They insist that industries stop the pollution of streams by constructing sewage plants or by combining forces with the municipalities in the study of the problem.

As has been so clearly emphasized by the author, separate digestion and the design of better digestion tanks have been brought about by scientific research, and the development of this method is merely a link in the chain of progress made possible by a working knowledge of the plants which have demonstrated these improvements in successful operation. Some of the advantages of separate sludge digestion may be classified as follows.

First.—Simplicity of Construction.—A separate sludge digestion plant is built of simple concrete structures. The digestion tanks are constructed above the water-table, and do not entail any unusual construction cost or expensive excavation.

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* Chairman, Wisconsin Highway Comm.; Pres., Jerry Donohue Eng. Co., Sheboygan, Wis.

Second.—This type of plant is operated mechanically. This is a decided advantage to the smaller communities, because it compels careful operation. The old type of sewage disposal plant, when once installed, was neglected and ignored as the municipal authorities paid little attention to operation. The mechanically operated plant requires attention; therefore, the process of sewage purification will go on more efficiently, and better results will be obtained.

Third.—Flexibility as to Expansion.—Either the sedimentation unit or the digestion unit may be enlarged and thus maintain a balance between them. If conditions should arise which would require an expansion of the digestion compartment, this can be accomplished by merely adding another digestion tank.

Fourth.—The separate sludge-digestion type of plant is better adapted to the collection of gases than most other types, as the gases can be trapped in the digestion tank and conducted to a gas burner, thus utilizing a waste product for heat and power and increasing the efficiency of the plant.

Fifth.—The period of digestion made possible by the application of heat to the sludge may be extended to a full twelve months, whereas in most types of plants the digestion process stops just as soon as the temperature drops below 50° Fahr. By adding heat to the sludge, the period of digestion can be extended and accelerated, resulting in a saving in capacity. In the middle western and northern climates where the temperature ranges from 30° below zero to 90° or 100° Fahr., above zero, the extension of the period is a decided factor in an economical selection of the type of plant best adapted to the conditions.

The cost of mechanical operation in separate sludge digestion plants is very small. The Hartford, Wis., plant was the first of this kind in the State of Wisconsin. The cost of power to operate this plant is $15 per month, or about 50 cents per day. This includes all power necessary for motor operation, pumping of sludge, and the lighting necessary in the building.

The total annual cost of operation of the plant at Antigo, Wis., including power, light, chemicals, etc., is 64½ cents per capita. From this cost, however, should be deducted the value of the reclaimed gas, which is utilized in heating the sludge in the digester. If the value of this gas is credited at the rate charged by the local utility company, it would reduce the cost of operation to about 18 cents per capita.

Separate sludge digestion tanks should be built with tight covers to prevent odors from escaping into the air and to permit the efficient collection of gases. The type of cover usually recommended is a concrete slab constructed so as to permit it to expand and contract with the temperature. This concrete slab, which is supported by the circular side walls and through the center by the same truss that carries the operating mechanism that agitates the sludge, rests upon a mastic joint extending the full thickness of the supporting wall. In the center of the supporting wall, the joint is keyed into the concrete. The depth of the key provides space for a crimped copper strip which extends around the full circumference. The joint is filled completely with asphalt which allows the copper strip to elongate under the stress of the moving concrete top, the copper being firmly held at both ends by its embedment in the concrete roof slab on the top and the concrete wall on the bottom. This form of joint makes the digester both water-tight and gas-tight.

When sludge is withdrawn from the digestion tank, the level of the liquid in this tank is lowered. This raises the question as to the advisability of having a gas container in series with the gas line so that after sludge is withdrawn, gas will follow back into the digestion tank to take the place of the liquid that has been removed. A gas container, with a capacity of one day's or two day's supply, would have the further advantage of stabilizing the flow of gas to the burners and of supplying gas during the time that sludge is being withdrawn, or while repairs are being made to the gas dome or digester.

It is predicted that the development of the next few years will include a combined type of plant of activated sludge with separate sludge digestion, using the better features of each type. This combination will enable smaller cities to consider activated sludge treatment, with the reduction of the sludge by separate digestion. This will largely solve the sludge problem for communities that experience difficulty in disposing of their sludge, for by digestion the volume is reduced 40%, and the residual sludge dries readily and makes a good fertilizer.

The Antigo plant seems to be producing approximately 1 cu. ft. of gas per capita per day. Assuming a conservative estimate of 0.8 cu. ft. per capita per day, the annual value of the gas based on the local utility rate would be about 45 cents per capita. The heating value of the gas, as determined by analysis, averaged 550 B. t. u. per cu. ft., so that a recovery of more than 400 B. t. u. per capita per day can be depended upon. This, reduced to coal at 12 000 B. t. u. per lb., is equal to 6 tons per year per 1 000 population.

KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).—In 1897, the Englishman, W. D. Scott-Moncrieff, carried out a most interesting experiment† in passing septic tank effluent through a series of nine trays placed one above another, each containing 6 in. of coke. No deposit had accumulated in 3 months of operation while, in the 8 or 10 min. required for passage through the series, a high degree of purification had taken place. This was attributed in part to the large proportion of liquefying bacteria in the septic sewage which converted much of the organic solids to a condition easily attacked by nitrifying bacteria. The development of these bacteria, in turn, was promoted by the oxygen absorbed by the sewage in dropping from one tray to the next; but the rapidity of purification was also attributed to the selective development, in each tray, of the particular bacteria specially adapted to their environment. Consequently, their activities were carried on under the most favorable conditions, just as division of labor in a well-organized industrial plant brings about the highest productive efficiency. Basing conclusions on "oxygen consumed" tests, Dr. Samuel Rideal found an average of 93%, and on the oxidation of nitrogen, he found that 91.6% purification was brought about thus in approximately 8 min. In commenting on these findings he stated that* "the nitrate has developed with extraordinary rapidity and to an extent that exceeds any other process."

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* San. Engr., Sanitation Comm., New York, N. Y.

† Described in the 17th Annual Report of the Provincial Board of Health, Ontario, and in *Engineering Record,* Vol. XXXVIII, p. 561.

The advantages of segregating the several steps of sewage treatment have been recently brought forward by Wynkoop Kiersted, M. Am. Soc. C. E.,† in a plea for so complete a separation of suspended solids by preliminary treatment that oxidation may be rendered easier and more rapid and secondary sedimentation avoided altogether. This is advocated as an ideal for which to strive.

In the activated sludge process, Martin points out the irrationality of utilizing compressed air simultaneously for agitation and aeration. The supply of air required for agitation and that for the supply of oxygen to the sewage are unrelated; and in the attempt to secure adequate treatment an excess for either one or the other of these purposes will ordinarily be used. Martin states,‡ "this difficulty would be avoided if the stirring and the aeration were effected independently." In connection with septic and Imhoff tanks, Dr. Rudolfs has stated that§ sedimentation and digestion, inasmuch as they are two distinctly separate processes, cannot be combined for the best possible results because they are unlike in principle. Mr. H. C. H. Shenton has expressed the same opinion.‖

These examples illustrate the trend toward a separation of the functions involved in sewage treatment in general, in order that each may be carried out under the most favorable conditions.

In sludge digestion as ordinarily practiced, there is a tank influent of fresh sludge with, perhaps, a percentage of ripe sludge for seeding. After entering the tank, which is usually built as a vertical cylinder, this material is intimately mixed with the other contents by circulation or by mechanical stirring. It has been found that this mixing process promotes bacterial activity with the formation and release of gas and, at the same time, retards the formation of scum. To make room for the inflowing sludge an equal volume of the contents of the tank is drawn off as supernatant or as digested sludge.

If the process is properly controlled for a sufficient period, the sludge thus withdrawn is dark in color, drainable, and inodorous; but as it comprises a part of the sludge that has entered the tank each day for, say, 40 days, representing the length of storage, it is evident that it cannot all be completely digested. The question arises as to whether or not a gradual progression of the sludge in a long horizontal tank of relatively small cross-section—provided with stirring apparatus to ensure a thorough mixing in that particular cross-section at a number of points—would not produce an equal result in less time or a more thoroughly digested product in the same time. It is probable that, with the varying condition of the sludge during the period between its entrance and discharge from the tank, there would be a corresponding variation in the specific type of bacteria, requiring different pH control and temperature for optimum results. In a tank of this kind it would seem fairly reasonable to approximate these requirements at perhaps three or four points.

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* *Journal,* Sanitary Inst., Vol. XIX, Pt. IV.

† *Municipal News and Water Works,* March, 1929.

‡ "The Activated Sludge Process," Chapter XXI, p. 249.

§ See p. 717.

‖ "Common Defects at English Sewage Disposal Works."

With the exception of control, the Merchantville-Pensauken tank, converted about 1924 from a septic tank to a digestion tank, complies in principle with the writer's suggestion.* As reconstructed, this tank is in the form of a baffled channel 6 ft. wide, 10 ft. deep, and 240 ft. long, giving a capacity of about 2 cu. ft. per capita for 11 000 persons. The sludge flows very slowly through this channel until it is drawn at the farther end, some time previous to which it has become black and thoroughly digested. The sewage flow averages about 1 000 000 gal. per day. In March, 1927, samples drawn at ten different points between the inlet and outlet ends showed that† "all samples after Station 5 were well digested." In a test made in February, 1927, the pH rose from 6.0 in the influent to 7.4 at Station 6 and 7.6 at the effluent. Between the sixth and eighth stations, gas evolution was very vigorous and the final product, after 9 months' retention, was black, granular, tarry in odor, and had a 75% reduction of biological oxygen demand. In a week or ten days of warm dry weather it will shrink from a depth of 13 in. to 4 in. on the drying beds and then can be heaped up and burned to a fine ash without additional fuel. According to Maj. M. J. Blew, Research Engineer, North East Sewage Treatment Plant, Philadelphia, Pa., the time of retention can be reduced about 20% by returning the tank supernatant back to the influent of the sedimentation tank, and it can be further reduced to 4 or 5 months by seeding with digested sludge, while by temperature control it can probably be cut to 6 weeks, or 2 months. The satisfactory results secured without pH control or artificial heating would undoubtedly be exceeded considerably with the adoption of such measures. In applying the principle of progressive digestion, two general types of construction are possible. By the first, the tank would consist of a long conduit slightly below the hydraulic grade, but with a gentle upward gradient to enable the released gases to collect in a dome at the outlet end. Baffles at the inlet would distribute the incoming sludge, seeded and limed, if necessary, and provision for pH control and for the removal of deposits, if formed on the rounded invert, by flushing or mechanical means, would be made at several points between the two ends of the tank.

By the second arrangement digestion would take place by steps in several separate tanks operated in series, each under independent control, the units of which could be of the "Boonton" type, as shown in Fig. 2, or of the usual Dorr type.

A preliminary digestion in Imhoff tanks followed by completion of the process in a relatively small separate tank, as advocated for large plants by Karl Imhoff,‡ M. Am. Soc. C. E., is in accordance with this idea. The sludge chamber in the Imhoff tank would be of smaller size and the partly digested contents pumped in a condensed form to the separate tank. Acid fermentation is avoided and optimum temperatures can be provided, all with minimum sludge capacities.

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* *Public Works*, June, 1924, p. 184, July, 1924, p. 213, November, 1927, p. 409, and December, 1927, p. 468.

† *Loc. cit.*, December, 1927, "Merchantville-Pensauken Sewage Treatment Plant," p. 470.

‡ "The Arithmetic of Sewage Treatment Works," by Karl Imhoff and Gordon M. Fair, p. 37.

Prüss has elaborated the same idea.* He computes the profit (after paying capital and operating charges) in liters of sludge gas per day per capita when selling at 6 pfennigs per cu. m. With single-stage digestion at either 12° or 25° cent., the surplus or profit is 0.2 liters, and 3.0 liters with two-stage digestion when heating the second unit, only, to 25° cent.; that is, if the surplus gas were sold—as is customary in Germany—the net profit with two-stage digestion would be fifteen times that with a single tank.

Whether the cost of providing more than two stages to the process would be justified is not known, but if there is an economic advantage in adding an auxiliary tank to the sludge chamber of an Imhoff tank there would appear to be a still greater economy in providing optimum conditions for all stages in a single tank near the ground surface.

Richard H. Gould,† M. Am. Soc. C. E. (by letter).—Discussions oftentimes contribute material and thoughts more valuable than the original paper. From the writer's standpoint, the discussions submitted in this case certainly are most interesting and instructive.

The comments on the paper come from engineers representing the interests that are bringing about the noteworthy advances in the art of sewage disposal. The discussion includes remarks by consulting and sanitary engineers, a research investigator, an experienced plant operator, and a representative from a firm supplying appliances used in sewage disposal. It is gratifying to realize that the improvements in sewage disposal methods are being followed so closely by people representing so many different points of view, and to know how each group recognizes its dependence one upon the other.

Mr. Eddy has traced the development of sewage disposal, and has pointed out the trend toward more elaborate and costly plants. He has described certain of the difficulties that this trend involves. Many of the plants now being constructed require highly technical supervision in their operation. The fact that it is difficult to secure operators with adequate training may perhaps be classified as a "growing pain" which may soon be overcome. The various sewage works associations, with the co-operation of some of the State boards of health and the educational institutions, are now making determined efforts to increase the technical knowledge and ability of sewage plant operators. Mr. Donohue makes the argument in this connection that, if equipment requiring expert supervision is installed, the necessary supervision will be supplied and the operation of the plant as a whole will benefit thereby.

It is to be expected that the cost of disposal works will be higher than was formerly necessary. In many cases the necessity for sewage disposal depends on the concentration of population, where means for the assimilation of sewage in the natural bodies of water are limited. The increase of population creates increased land values and may make it impossible to secure isolated sites, where some of the cruder methods of disposal might be practiced. The trend is, therefore, toward the construction of plants on a limited area capable of bringing about a high degree of purification, and doing this without the production of objectionable odors that will lower adjacent property values.

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* "Fortschritte in der Ausfaulung von Abwasserschlamm," von Dr. Ing. Max Prüss, 1928.

† Chf. Engr., Ward's Island Sewage Treatment Works, New York, N. Y.

The demands made in many cases require a more or less complicated plant, and the amounts invested certainly warrant highly trained technical supervision. It would be false economy indeed, to withhold proper compensation for such service. After all, the fact that complete disposal of sewage can usually be secured at a cost of about $1 or $2 per capita per year, permits little criticism of such an expenditure for so important a necessity.

The discussions by Mr. Besselievre and Mr. Allen bring out some of the differences of opinion that are common in sewage disposal practice. Mr. Allen thinks there may be some advantage in digestion in separate stages, and points out the disadvantage of mechanical stirring, in that some of the fresher sludge is discharged with the more thoroughly digested material. It seems that more data are necessary to establish definitely the fact that there are practical advantages in several separate digestion stages.

The action in a digestion tank, as described by Dr. Rudolfs, where the solids in a state of gasification are constantly rising and falling, depending on whether or not bubbles of gas are attached to the particles, does, in effect, tend to separate the digestion processes to a certain extent; that is, the particles undergoing active gasification tend to remain in the upper part of the tank, while those which have become inert work their way toward the bottom.

It would be interesting to compare the effectiveness of the gentle stirring caused by a mechanical contrivance, with the stirring action of these rising gases of digestion. It will be recalled that many years ago, Dr. Imhoff used this argument among others, in advocating the use of deep tanks, when, as in this case, there would be better stirring action owing to the increased amount of gas produced per square foot of area.

The speculation of engineers, the researches of scientists, and the mechanical ingenuity of manufacturers are extremely valuable; but, after all, the results secured in operation are the most important. So it is that concrete results of operation, such as that given by Mr. Downes, are especially welcome.

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Paper No. 1743

STUDIES OF SHEAR IN REINFORCED CONCRETE BEAMS*

By T. D. Mylrea,† M. Am. Soc. C. E.

With Discussion by Messrs. Edward Godfrey, C. A. P. Turner, H. E. Eckles, Jacob Feld, Hardy Cross, P. Wilhelm Werner, and T. D. Mylrea.

Synopsis

Although much has already been written on the subject of shear in reinforced concrete and although design practice has become almost uniformly standardized, there still remain a number of unsettled problems in reinforced concrete design. Standardization, however many its advantages, may easily become synonymous with stagnation. In many respects it can be said that, in the interest of getting structures built, unsettled points have simply been allowed to drop. "This apparently works all right" has been answer enough. Recently, both structural steel and reinforced concrete have begun to feel the benumbing effects of standardization, and some rather drastic revisions have been made in structural standards. Whenever such revisions are made it is of prime importance to re-examine the basis of structural theory to check the assumptions on which it is founded, and to weigh carefully the supporting evidence of experiment.

The writer has attempted to do this in the case of shear in reinforced concrete. In so doing he does not wish to be regarded as a reactionary, but merely as one taking an inventory of stock, for he is satisfied that careful study will show that no serious consequences will follow the raising of certain allowable unit stresses on condition that such increases are not haphazard and that adequate provision is made for all details. In fact, many of the suggestions are in the nature of a survey of the possibilities of the future. It is necessary first to consider briefly the derivation of the usual formulas in order to take up a detailed discussion.

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* Published in January, 1929, *Proceedings*.

† Prof., Bldg. Constr., Carnegie Inst. of Tech., Pittsburgh, Pa.

Notation

The following notation is used throughout the paper:

n = the ratio of the moduli of elasticity, E_s and E_c.
E_s = the modulus of elasticity of steel.
E_c = the modulus of elasticity of concrete.
N = the number of reinforcing bars in a distance, B.
B = the length of a tension crack measured perpendicular to the bent of bars, assuming that it were possible for the crack to extend to the center of compression (which it never can do).
j = the ratio of the lever arm of the resisting couple to the depth, d.
d = the effective depth of a reinforced concrete beam.
s = the horizontal distance between stirrups or points at which reinforcing bars are bent up.
V' = an external shear resisted by reinforcing steel.
P = the average pull per stirrup.
a = the bent bar spacing, perpendicular to their direction.
α = the angle of slope of the bent-up bars.
A = the length of the tension crack, assuming the direction of the crack to be at 45° and that it extends to the center of compression.
v = unit shear.
b = width of beam.
v_a = average unit shear.
v_m = maximum unit shear.
c = diagonal compression.
T = tension in a bar.
u = unit bond stress.
Σo = the total perimeter of the reinforcing bars.
c_h = horizontal component of c.
c_v = vertical component of c.
w = uniformly distributed load.
k = ratio of depth of neutral axis to effective depth.

Stresses Induced in Web Reinforcement

Just as longitudinal reinforcement is provided where longitudinal tension occurs in a reinforced concrete beam so, in the web, reinforcement should be provided to resist diagonal tension. The two phenomena are in no wise different from each other, the curve of tension being continuous along a stress trajectory. Yet confusion has often obscured this point, diagonal tension being regarded as a phenomenon of mysterious origin and much to be feared. If reinforcement is properly placed so that it can neither slip nor crush the concrete, and is provided in sufficient quantity, tension in a diagonal direction is no more to be feared than tension in any other direction. Such reinforcement may be either inclined or vertical and each requires special consideration.

Let Fig. 1 (a) represent a beam in which the web reinforcement consists of stirrups. Before cracking occurs, the displacement of the stirrups due to flexure only will be as shown in Fig. 1 (*b*). At the top they will be closer together and at the bottom they will be farther apart, but there will be no change in total length. Due to shear only the detrusion of the concrete permits a movement of one stirrup vertically with respect to its neighbor, but still there is no change in length. (See Fig. 1 (*c*).) Therefore, before cracking takes place, neither flexure nor shear will cause any stress in vertical stirrups. This is confirmed in the experiments* of A. N. Talbot, Past-President, Am. Soc. C. E., in some of which there even seemed to be evidence of slight compression in the stirrups when a low load was first applied. In other tests investigators have apparently detected slight tension from the very start. Much unfruitful argument has resulted concerning this point. Before cracking, a reinforced concrete beam behaves much as any other homogeneous beam. Since the shear is not uniformly distributed from top to bottom in such a beam, it is possible that below the neutral axis a vertical compression might exist, while above this axis there might be a slight tension. An embedded foreign body, such as a vertical stirrup, might thus register either tension or compression to a slight degree, depending on whether the measurement was made above or below this axis. The important feature, however, is what happens after the concrete cracks.

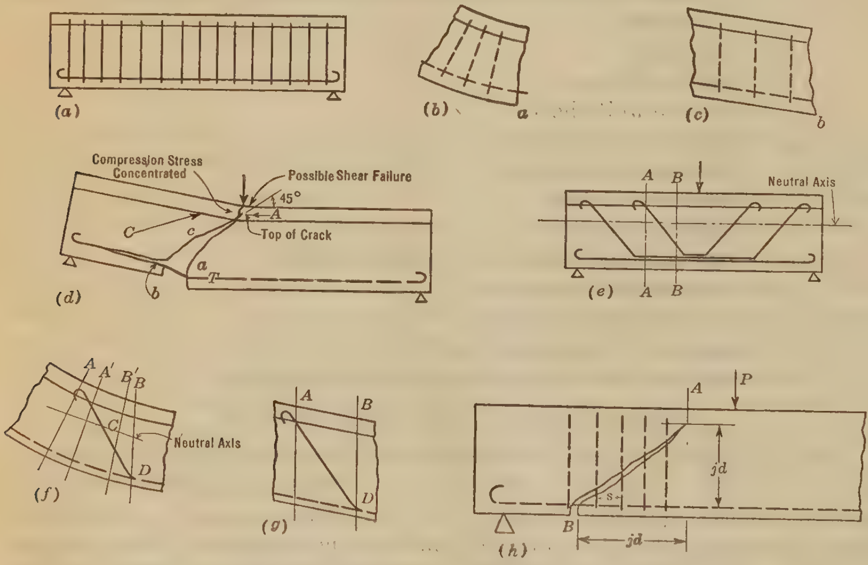

FIG. 1.—ELEMENTS OF VARIOUS REINFORCED CONCRETE BEAMS.

In a beam that is once cracked, diagonal tension failure, unresisted, would soon follow, as shown in Fig. 1 (*d*); but if the projecting lower cantilever, *a*, and the reinforcement at *b* were secured to the part marked *c*, failure would be greatly delayed. This is the function of vertical stirrups.

Consider the diagonal web reinforcement shown in Fig. 1 (*e*). Before cracking occurs, and under flexure alone, Sections *A* and *B* are moved relative to each other, as shown in Fig. 1 (*f*). Above the neutral axis there would be compression in the diagonal bar, due to the shortening of the length, *AC*, while tension would be found below in the length, *CD*. Due to shear alone (see Fig. 1 (*g*)), Section *B* being moved vertically in relation to Section *A*, the whole length, *AD*, is increased. Thus, as a result of the combined action of flexure and shear, tensile stress might be present in diagonal reinforcement, below the neutral axis at least, before any cracks appear, and such reinforcement would serve to toughen a beam under low loads better than vertical stirrups. Since adhesion may be considered perfect before cracks appear, the stress in the steel would not be greater than n times the stress in the surrounding concrete, in which, $n = \frac{E_s}{E_c}$. After the beam has cracked, the bars extending diagonally across the crack serve the same function in delaying failure as the vertical stirrups (see Fig. 1 (*a*)).

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* *Bulletin No. 28*, Univ. of Illinois Eng. Experiment Station, p. 24.

FORMULAS FOR WEB REINFORCEMENT

Let Fig. 1 (*h*) represent a half length of a beam subjected to that part of a constant external shear, V', not "taken by the concrete". It is assumed that a crack will take place at an angle of 45°, as shown, even though it cannot extend to the center of compression. Let a section, *A-B*, be passed along the crack. Then, in performing their part of the duty of maintaining, as nearly as possible, the homogeneity of the beam, the stirrups merely suspend the right side of the beam from the left. The load, V', is divided between N-stirrups, and (from Fig. 1 (*h*)):

$$N = \frac{j\,d}{s} \quad \text{.........................} (1)$$

Therefore, if P is the average pull per stirrup,

$$P = \frac{V'}{N} = \frac{V'\,s}{j\,d} \quad \text{.........................} (2)$$

In Fig. 2 (*a*), the web reinforcement consists of diagonal bars. Here, the load, V', is carried by the vertical components of N-bars. From Fig. 2 (*a*):

$$a = s \sin \alpha \quad \text{.........................} (3)$$

$$A = \frac{j\,d}{\cos 45^\circ} \quad \text{.........................} (4)$$

$$B = A \cos (45^\circ - \alpha) \quad \text{.........................} (5)$$

and,

$$N = \frac{B}{a} \quad \text{.........................} (6)$$

The vertical component of one bar $= P \sin \alpha$. Therefore:

$$V' = N\,P \sin \alpha \quad \text{.........................} (7)$$

Substituting in Equation (7) the values of N, A, and B (from Equations (3), (4), (5), and (6)), and reducing,

$$V' = \frac{P\,j\,d\,(\cos \alpha + \sin \alpha)}{s} \quad \text{...................} (8)^*$$

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* For another proof, see *Technologic Papers No. 314*, U. S. Bureau of Standards, p. 392, by Messrs. W. A. Slater, A. R. Lord, and R. R. Zipprodt.

Equations (2) and (8) may be transposed, to suit the requirements, so as to give (1) the vertical external shear that may be carried by the reinforcement in a beam of depth, d; (2) the pull per stirrup or bent bar; and (3) the required spacing. For the special case in which the slope of the reinforcement is 45°, Equation (8) becomes:

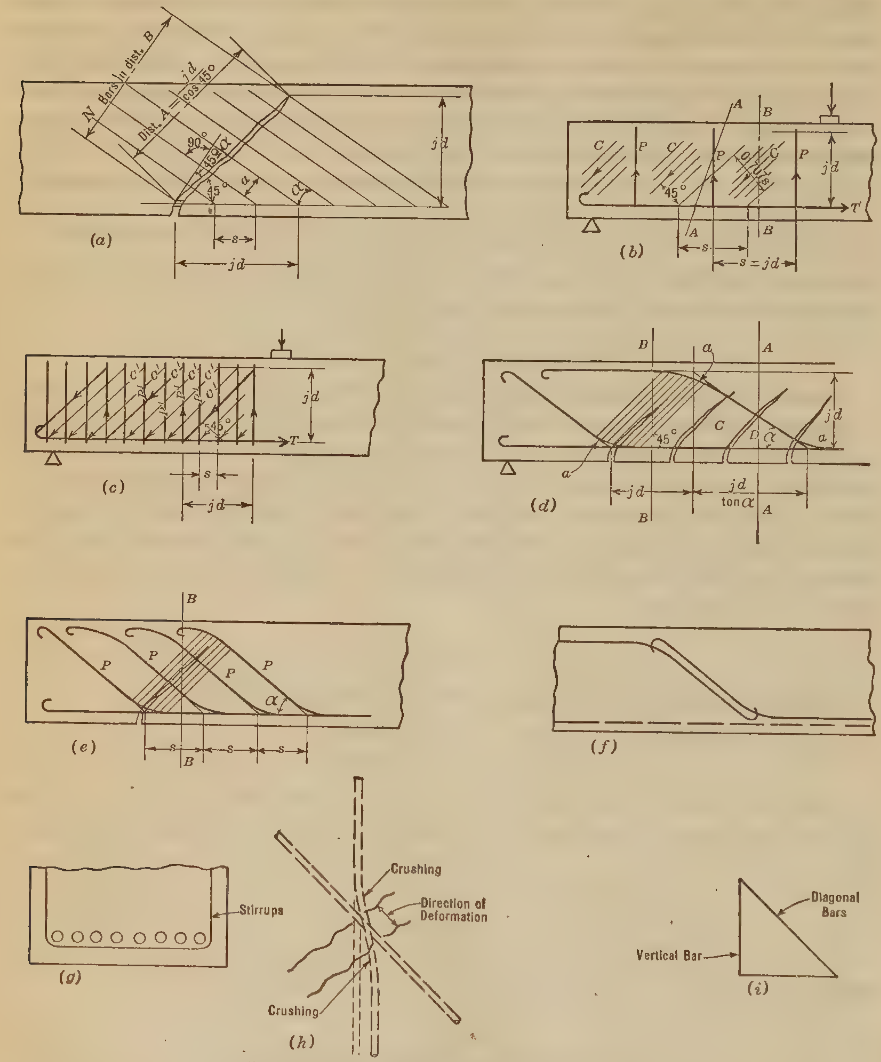
N Bars in dist. B
Dist. A = jd/cos 45°
90°
45°
α
a
jd
s
(a)
A
B
C
P
T
45°
s = jd
(b)
c'
45°
(c)
B
a
A
C
D
jd/tan α
(d)
P
(e)
(f)
Stirrups
(g)
Crushing
Direction of Deformation
(h)
Diagonal Bars
Vertical Bar
(i)

FIG. 2.—WEB REINFORCEMENT DETAILS.

$$V' = 1.414\, P\, j\, \frac{d}{s} \quad \text{.......................} (9)$$

$$P = 0.707\, \frac{V'\, s}{j\, d} \quad \text{.......................} (10)$$

or,

$$s = \frac{1.414\,P\,j\,d}{V'} \quad \text{(11)}$$

A comparison of Equation (11) with the value of s obtained by Equation (2), shows that, all other conditions being equal, bars of the same size spaced the same distances apart horizontally will resist 1.4 times as much shear when inclined at 45° as when they are vertical. Expressed in a slightly different way, to carry the same shear, bars at 45° might be spaced 1.4 times as far apart as vertical stirrups of the same cross-sectional area. Since, however, such bars are 1.4 times as long as vertical bars there is no saving in weight. In Equations (1) to (11), a uniform load may be provided for by using the average shear in the length, jd.

TRUSS ANALOGY

In Europe it is common practice to conceive the web reinforcement as the tension web members of a truss, the concrete acting in lieu of compression members inclined at 45 degrees. With vertical stirrups the imaginary truss would thus be of the Howe type, while with diagonal reinforcement it would be of the Warren type. If the bars are closely spaced the trusses are of the multiple-intersection Howe or Warren type, respectively, the compression being assumed always to act at an angle of 45 degrees.

In Fig. 2 (b), the stirrups are spaced at distances equal to the effective depth of the beam. By passing Section A-A it is seen that $P = V'$. If intermediate stirrups are inserted, as in Fig. 2 (c), the load, P, is divided among them in the ratio of $\frac{s}{j\,d}$. Then, $P = V' \frac{s}{j\,d}$, the formula developed for the member considered as a beam. (See Equation (2).)

In Fig. 2 (d), the distance between successive bends is $j\,d + \frac{j\,d}{\tan\alpha}$. Again, passing a section, A-A, and analyzing as a truss, the stress in any bent bar is:

$$P = \frac{V'}{\sin\alpha} \quad \text{(12)}$$

If intermediate bent bars are used at equal spaces, s, as shown in Fig. 2 (e), the total stress, $\frac{V'}{\sin\alpha}$, is divided among N-bars, and,

$$N = \frac{j\,d + \dfrac{j\,d}{\tan\alpha}}{s} \quad \text{(13)}$$

The stress in any one bar is, therefore:

$$P = \frac{\dfrac{V'}{\sin\alpha}}{\dfrac{j\,d}{s}\left(1 + \dfrac{1}{\tan\alpha}\right)} = \frac{V'\,s}{j\,d\sin\alpha\left(1 + \dfrac{1}{\tan\alpha}\right)} = \frac{V'\,s}{j\,d\,(\sin\alpha + \cos\alpha)} \quad \text{(14)}$$

which is the same as Equation (8). It makes no difference, therefore, whether the member is conceived as a beam or as a truss, the resulting formulas for the web reinforcement are the same. To be sure, in the truss analogy, bond between the longitudinal steel and Blocks C and D (Fig. 2 (d)) is neglected, but this factor is probably over-estimated, particularly at higher loads.

The truss analogy, however, makes evident a number of tendencies requiring caution in design. For example, unless the concrete is to be dangerously over-stressed at Points a (Fig. 2 (d)), long bends are seen to be imperative. At the same time it is evident from this analogy how careful the designer should be to extend the upper part of the bar back into the concrete far enough to provide an effective anchorage. It shows, too, that such a splice as that indicated in Fig. 2 (f) might be questionable, especially when the splitting effects of the hooks are taken into consideration. It shows how necessary it is to anchor the web reinforcement at both top and bottom strongly enough to develop its full tensile strength. No one familiar with structural steel design would think of providing web members of sufficient area to take the stress, and then only partly fastening them at the ends. Unless web members are fully developed above the cracks they are ineffective.

Furthermore, since a web member comes into play only after a beam has cracked, and since the diagonal compression (see Fig. 2 (b) and Fig. 2 (c)) is considered uniform through the whole thickness of the web, the vertical component must be taken up at frequent intervals across the web. Vertical stirrups, therefore, enclosing a large number of bars, as shown in Fig. 2 (g), are practically useless. The flexible lower reach of the stirrup can never prevent the inner bars from tearing out through the bottom of the beams at high loads, nor even prevent the loosening of the bond on the longitudinal bars at low loads. (See Fig. 1 (d) at Point b.)

COMBINATIONS OF BENT BARS AND STIRRUPS

Quite commonly there are not enough bars available for bending to resist shear, and vertical stirrups are introduced to assist. Generally, the same unit tensile stress is used in computing both bent bars and stirrups. Such a procedure, however, is not warranted.

It was shown that diagonal bars are stressed from the beginning. In concrete with an assumed tensile strength of 300 lb. per sq. in., and $n = 10$, this steel will be subject to a tension of $300 \times 10 = 3\,000$ lb. per sq. in., before the crack occurs. Careful observations of cracks will show the deformations to be as depicted in Fig. 2 (h). The relative extension of diagonal and vertical bars, therefore, would be somewhat as shown in Fig. 2 (i), producing relatively higher stresses in the diagonal bars than in the stirrups. This ratio would be 1.4 to 1 if it were not for the tendency of the stirrups to crush into the concrete (see Fig. 2 (h)). Neglecting this crushing and assuming the maximum allowable steel stress to be 16 000 lb. per sq. in., the stress in the vertical steel is only $\frac{16\,000 - 3\,000}{1.4} = 9\,300$ lb. per sq. in. when the stress in the diagonal steel is 16 000 lb. per sq. in. The unit stress developed in the stirrups being only 9 300 lb. per sq. in., they will have to resist their part of the shear at a unit stress not exceeding this value when the bent bars are stressed simultaneously to 16 000 lb. per sq. in. This, of course, becomes of much more importance when dealing with the high stresses that can reasonably be anticipated to be the common practice of the near future.

It has frequently been observed that a beam with web reinforcement consisting of a combination of bent bars and stirrups usually develops somewhat higher shear strength than one reinforced with either bent bars or stirrups alone. In the beam thus reinforced the stirrups probably restrain the bent bars from breaking out through the sides of the beams under heavy loads.

From these considerations it may be expected that, in a beam subjected to an increasing load, the web reinforcement would act somewhat as follows: The diagonal bars would be stressed from the beginning. Simultaneously with the occurrence of a crack they would receive a large increment of stress, and the vertical stirrups would also come into action; but the stress in the diagonal bars would be greater than that in the stirrups. If the conditions are such that both bent bars and stirrups can be fully developed, the stresses in both will increase, but at a more rapid rate in the diagonals than in the stirrups. After the diagonals reach the elastic limit, they will deform without much increase in stress for a time, after which the stress will again increase, but at a slower rate. Meanwhile, the stress in the verticals will increase rapidly until in them, too, the elastic limit is reached. Therefore, until their usefulness is ended, the stress in the stirrups will always lag behind that in the bent bars. Such a lag was observed in the tests of the Emergency Fleet Corporation.*

COMPARATIVE DEFLECTIONS

The deflection of beams with different web reinforcement might also be expected to vary. Since it requires a greater vertical movement at a crack to cause a given stress in vertical stirrups than in bent bars, then for a given steel stress a greater deflection will result in a beam reinforced with stirrups than in one of equal web strength reinforced with bent bars. If both bent bars and stirrups are used, an intermediate deflection will result, its magnitude depending on the proportion of each type of reinforcement. Thus, beams of equal strength may deflect differently. This is of no practical importance until great deflections occur, when the added increment of deflections permitted by the stirrups may result in earlier compression failure due to the concentration of compression stress near the top of the beam.

Some work has been done in an attempt to predict deflections. Turneaure and Maurer† have developed an analytical treatment of the subject and G. A. Maney, M. Am. Soc. C. E., has made an interesting study of test data.‡ It is rare that deflections need to be known in advance, and, except when limited to deflection under loads producing comparatively low stresses, attempts to predict them usually fail. In addition to the influence of the type of web reinforcement this uncertainty is probably due, in a large degree, to the fact that slip of any of the reinforcement strongly influences the results, and, as has been shown,§ slip occurs early in the load history of most beams An attempt to treat, analytically, an action so dependent on the approximate homogeneity of the constituent parts cannot but fail when an unknown element of slip occurs.

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* *Technologic Paper No. 314*, U. S. Bureau of Standards.

† "Principles of Reinforced Concrete Construction," Third Edition, 1919, Chapter VI.

‡ *Proceedings*, Am. Soc. for Testing Materials, 1914, p. 310.

§ "Bond and Anchorage in Reinforced Concrete," *Journal*, Western Soc. of Engrs., January, 1926.

A careful study of tests in which end slip as well as deflections has been observed, such as the tests of Bach,* seems to indicate that until end slip begins, like beams, reinforced alike, will deflect alike. It may be anticipated that when design reaches the stage where such uncertain elements as end slip are eliminated, deflections can be predicted accurately.

Proportion of the Shear "Not Taken by Web Steel"

It was found early that, if the total external shear, V, were used in Equation (2), the computed stress in stirrups often exceeded the known ultimate strength of the steel, from which it was concluded that other influences were at work in resisting shear. It was suggested† that part of the shear is carried by the uncracked concrete at the top of the beam, and that,

> "The actual proportion taken by the stirrups has not been established, but it seems that the stress cannot be more than two-thirds or three-fourths of the amount obtained by the formula."

It then became common practice to use only two-thirds of the total vertical shear in computing web reinforcement. Later, it was discovered that when the loading was such as to produce a greater shear at one end of a beam than at the other, the two-thirds rule gave apparently anomalous results, and practice has largely shifted to allowing a fixed value for the percentage of shear "taken by the concrete" (that is, shear not discernible in the steel). A value is generally taken that is a definite proportion of the ultimate compressive strength of the concrete cylinders when tested at the age of 28 days. In recent practice this proportion has been made to depend on the end anchorage of bars.

Confirmatory test data bearing on this phase are not plentiful. Some measurements have been made at the University of Wisconsin.‡ More recently, the results obtained by the Emergency Fleet Corporation§ have been published, and the University of Illinois Engineering Experiment Station has completed experiments‖ extending over a period of years, which add much to the available data.

From other tests made at the University of Illinois,¶ it appears that, even up to the ultimate load, the ratio of measured to computed stress in stirrups does not exceed 65 or 70%, whereas in the longitudinal steel in the same beams the ratio was 85%, or more. Of course, on a bar encased in concrete, an extensometer does not measure the maximum unit deformation at a crack, but only the average between two points, and along a large part of the gauge length the steel stress is decreased by the concrete; but if this average is less in one set of like measurements than it is in another, it is only fair to assume that the maximum is also less in the first set. From this it may be concluded that, if the total vertical shear is used in solving Equation (2), the actual stress in stirrups in beams as ordinarily designed is less than the computed stress.

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* *Deutscher Ausschuss für Eisenbeton,* Heft 38, 1917, by Dr. C. Bach and Prof. O. Graf.

† "Tests of Reinforced Concrete Beams," by A. N. Talbot, Past-President, Am. Soc. C. E., *Bulletin No. 29,* Univ. of Illinois Eng. Experiment Station, 1909, p. 72.

‡ *Bulletin 197,* Univ. of Wisconsin.

§ *Technologic Paper No. 314,* U. S. Bureau of Standards.

‖ *Bulletin No. 166,* Univ. of Illinois Eng. Experiment Station.

¶ Master's thesis, by S. Uchimera, Univ. of Illinois, 1922.

It is uncertain just to what extent other factors enter, but concrete capable of resisting compression should also be capable of resisting shear. Therefore, all the concrete above the crack (see Fig. 1 (*d*) and 1 (*h*), and Fig. 2 (*a*) to 2 (*d*)), undoubtedly offers considerable resistance to vertical shear, and, hence, under the usual design stresses an allowance for this effect is permissible.

The amount of the allowable reduction is still problematical. The deformation of a cracked beam is equivalent to a rotation about some point, *A* (Fig. 1 (*d*)), and the stirrup nearest the bottom of the crack would be strained and, consequently, stressed more than stirrups crossing the crack nearer the center of rotation. Nevertheless, the experiments quoted show that even in the stirrup that is stressed most the computed tension is more than the actual; and hence it would seem quite justifiable in ordinary work to assume that, if all the shear resisted by the concrete above the cracks were conceived as being uniformly spread over the whole cross-section of the beam, the average intensity thus found could be represented by some such value as one-fiftieth of the cylinder strength of the concrete. In the day when high shear values will be allowed this reduction will not be of great consequence, and it will be better to neglect it entirely. In fact, it will then be quite possible that, owing to the gradual time yield in shear of the concrete above the crack, the full shear, *V*, must be taken by the web reinforcement, and will be more nearly divided equally among the web bars crossing the crack.

Shear "Taken by the Concrete"

The phrase, "the shear taken by the concrete", has been confusing to many engineers. The preceding explanation is the one generally accepted. The laws of statics show, however, that the concrete may take all the shear, or only a part of it, depending on the form and distribution of the web reinforcement. For example, it is evident that on Section *B-B*, in Fig. 3 (*a*), the concrete resists all the vertical shear, and this capacity to resist shear is shared by the concrete both above and below the crack. A designer working in materials other than reinforced concrete would not hesitate to use two diagonal struts to carry a given diagonal compression, and he would not worry about a crack between them, providing neither strut could fail by buckling. Diagonal cracks, therefore, do not interfere with the capacity of the concrete in a beam to resist diagonal compression or vertical shear.

At the upper and lower surfaces of the beam, as well as at the upper and lower surfaces of the crack, the intensity of the shear will be zero, with a maximum between, as shown in Fig. 3 (*a*). The only requirement of statics is that the sum of the areas, *A* and *B*, at any section, *A* or *B*, shall equal the total vertical shear, *V*. The opinion is sometimes expressed, too, that the concrete below the neutral axis cannot resist horizontal shear because of the cracks. A consideration of Fig. 3 (*b*) shows that this is not true. It is evident that between cracks the stirrups do not resist horizontal shear, and across the crack they act vertically in tension only. Between cracks (Fig. 3 (*b*)), the concrete must resist all the horizontal shear occasioned by any difference in tension in the longitudinal reinforcement. Since close to the reinforcement this shear is directly dependent on bond, its distribution is similar to that of the bond stress, as shown at *C*, Fig. 3 (*b*),* while farther up it assumes more nearly the parabolic distribution shown at *D*. If the bond should break down entirely, the stress in the horizontal steel would be the same at both cracks, there would be neither horizontal nor vertical shear in the block of concrete between cracks, and all the vertical shear on Section *B* would be carried by the concrete above the top of the crack, as at *E*.

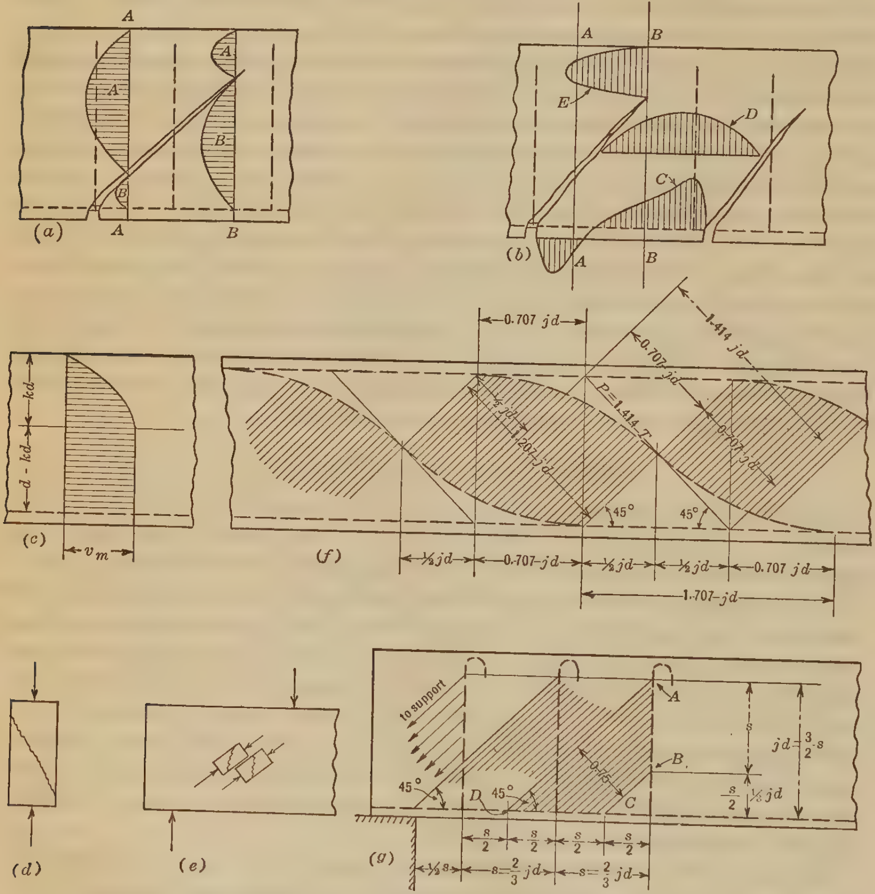
A
A
A
B
B
(a)
A
B
A
B
E
D
C
(b)
A
B
kd
d - kd
(c)
v_m
0.707 jd
1.414 jd
0.707 jd
½ jd
1.207 jd
P=1.414 T
0.707 jd
45°
45°
(f)
½jd
0.707 jd
½jd
½jd
0.707 jd
1.707 jd
(d)
(e)
to support
A
B
s
$jd=\frac{3}{2}s$
45°
D
45°
0.75
C
$\frac{s}{2}$
⅓ jd
(g)
⅓ s
$s=\frac{2}{3}jd$
$s=\frac{2}{3}jd$
$\frac{s}{2}$
$\frac{s}{2}$
$\frac{s}{2}$
$\frac{s}{2}$

FIG. 3.—STRESS DIAGRAMS.

While it is evident that when stirrups alone are used the concrete will have to resist the total vertical shear, *V*, on any vertical section, it is easily shown that when the web reinforcement consists of bent bars, the concrete may resist all or only part of the shear on the section. For example, at Section *B*, Fig. 2 (*d*), the concrete resists all the vertical shear, while on Section *A*, owing to the sharpness of the bends and to the long distance between bends, the truss analogy would indicate that the blocks, C and D, carry but little diagonal compression, and, hence, the external shear is resisted almost entirely by the vertical component of the tension in the steel. If, however, the bars of Fig. 2 (d) were spaced more closely together, as in Fig. 2 (e), and a vertical section were then passed, it will be seen that the shear on Section B would be divided between the vertical component of the tension in the steel and the vertical component of the diagonal compression.

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* *Journal*, Western Soc. of Engrs., January, 1926, p. 11.

Because of this division of the shear on a vertical section between the vertical component of the diagonal compression and the closely spaced diagonal web reinforcement, it has sometimes been presumed that the value of V' in Equations (8) to (11), might be much reduced. This is not the case, however. The usual formulas, Equations (8) to (11), developed for the critical section along a crack sloping at 45°, are quite consistent with the analysis on a vertical section.

In Fig. 2 (d), at Section B, the truss analogy would indicate that for a distance, $j\,d$, the whole vertical shear is resisted by the concrete, whereas at Section A, for a distance, $\frac{j\,d}{\tan\alpha}$, it is resisted entirely by the vertical component of the steel stress. When $\alpha = 45°$, then $\frac{j\,d}{\tan\alpha} = j\,d$. If the bent bar could be entirely developed, and if the diagonal compression could be carried trusswise down a single line from bend to bend, the bends might be spaced at distances $2\,j\,d$ apart. This result, however, may also be obtained from Equation (11), if V' is made equal to $0.707\,P$.

Thus, it is seen that in a beam reinforced with stirrups, the concrete resists all the shear on any vertical section, while in a beam with web reinforcement sloping at 45° and spaced not more than $j\,d$ apart, the concrete resists one-half the shear on any vertical section, and that this condition is quite consistent with the usual analysis based on a section passing diagonally along a crack. This statement will not confuse any one familiar with the usual method of analyzing double intersection Warren trusses. It will be seen, too, that the ordinary shear formulas include the "indirect inclined compressions" mentioned by Dr. Faber.*

NOMINAL DISTRIBUTION OF THE SHEAR

From Fig. 3 (a) and Fig. 3 (b), the shear distribution is seen to be very complicated, and it is impossible to state with exactness either the location or magnitude of its maximum value. Still, it is desirable to have some knowledge of relative shear intensities, and this can best be obtained in terms of the average shear on the section. Then:

$$v_a = \frac{V}{b\,d} \qquad \text{(15)}$$

The attempt is often made to find an expression for v_m, as follows: If a rectangular beam is assumed to be made of a concrete capable of taking shear but not tension, then between the neutral axis and the longitudinal steel the horizontal shear will be constant. Above the neutral axis the curve of shear intensity will follow a parabolic arc, as shown in Fig. 3 (*c*). The total shaded area must equal V, hence,

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* "Reinforced Concrete Beams in Bending and Shear," by Dr. Oscar Faber.

$$V = v_m (1 - k)\, b\, d + \frac{2}{3}\, v_m\, b\, k\, d = v_m \left(1 - \frac{1}{3}\, k\right) b\, d = v_m\, b\, j\, d \,..(16)$$

Since j is greater in T-beams than in rectangular beams, this formula automatically takes cognizance of the small shear carried to the support by the overhanging parts of the flange. It is obviously impossible, however, for the shear distribution to be as shown in Fig. 3 (*c*), where the concrete is cracked, and, in the region where no cracks exist, it must be in the form of the full parabolic arc. Except that it fits in with theory based on the usual assumptions, Equation (16) seems to have no special advantage over Equation (15), which is certainly easier to use and conveys as much to the mind.

DIAGONAL COMPRESSION

Considering the member subjected to flexure to act as a homogeneous beam, two sets of stress trajectories, intersecting at right angles, may be traced on the side, one of which delineates diagonal tension and the other diagonal compression. At corresponding points the magnitude of these stresses is equal. Considering the member as a trussed system, Fig. 2 (*b*) to Fig. 2 (*e*) plainly indicates the diagonal compression, and it remains to determine the magnitude of this stress.

When vertical stirrups are used, spaced a distance, $s = j\, d$, apart (see Fig. 2 (*b*)), the vertical component of the diagonal compression is equal to the pull, P, on the stirrups. The magnitude of the diagonal compression, C, is, therefore, 1.4 P; and acting on a width of 0.707 s, its intensity is:

$$p = \frac{1.4\, P}{0.7\, s\, b} = 2\, \frac{P}{s\, b} = \frac{2\, V'\, s}{j\, d\, s\, b} = \frac{2\, v\, b\, j\, d\, s}{j\, d\, b\, s} = 2\, v \,\ldots\ldots\ldots..(17)$$

Similarly, when the stirrups are more closely spaced, and $P' = V' \dfrac{s}{j\, d}$, the increment of diagonal compression, c', Fig 2 (*c*) contributed by each stirrup, is equal to $1.4\ V' \dfrac{s}{j\, d}$. Acting on a width, 0.707 s, the intensity of the diagonal compression is, again,

$$p = \frac{1.4\ V'\ s}{j\ d\ 0.707\ b\, s} = 2\, v$$

That is, when vertical stirrups alone are used the intensity of the diagonal compression is equal to twice the average vertical shear at the section under consideration.

Again, when only diagonal web reinforcement (see Fig. 2 (*e*)) is used, with such a spacing, s, that adjoining diagonal compression bands just touch along their edges, the vertical component of the diagonal compression equals $P \sin \alpha$. The magnitude of the diagonal compression is, therefore, $1.4\, P \sin \alpha$. Acting on a width, 0.707 s, the resultant intensity is:

$$p = \frac{1.4\, P \sin \alpha}{0.7\, s\, b} = \frac{1.4 \times 0.7\ V'\, s \sin \alpha}{0.7\, s\, b\, j\, d} = \frac{1.4\, v\, b\, j\, d \sin \alpha}{b\, j\, d} = 1.4\, v\, b \sin \alpha ..(18)$$

When $\alpha = 45°$, this expression becomes:

$$p = 1.4\,v \times 0.7 = v \quad\text{.....................(19)}$$

That is, when bent bars are used, the intensity of the diagonal compression is only one-half as great as when vertical stirrups alone are used; and when these bars are bent at 45°, the intensity is equal to the average vertical shear.* Thus, while the practice of placing a limit on the unit vertical shear as a means of controlling the diagonal tension is unnecessary, if sufficient completely developed steel is used to resist the diagonal tension, such a limit may very properly be used to prevent the occurrence of excessive diagonal compression stresses.

It has already been stated that, in concrete, failure by pure shear is difficult to produce. "Compression" failure of a brittle prismatic specimen, however, is really shear failure (see Fig. 3 (*d*)), and allowable unit compression values are usually determined by dividing the ultimate unit "compressive" strength of prismatic test specimens by a factor of safety. In the case of diagonal compression in a beam (see Fig. 3 (*e*)), the condition is analogous to that in a prismatic specimen, and if the allowable diagonal compression stress is limited by values determined from prismatic test specimens, a failure in pure shear cannot occur. The very fact that compression is inclined is due to the inclusion of shear as a factor. Thus, if the diagonal tension and diagonal compression are effectively taken care of, shear need be given no further consideration, its principal use being simply as a measure of the diagonal compression. (See Fig. 2 (*b*) and Fig. 2 (*d*).)

Spacing of the Bends in Web Steel

The writer has stated that if bars bent at an angle of 45° could be entirely developed, and if the diagonal compression could be carried "trusswise" down a single line from bend to bend, the bends might be spaced at distances of $2\,j\,d$ apart. Under such circumstances there are parts of the concrete not used in resisting diagonal compression, and the beam is not developing its full capacity in shear. Some modification of spacing, therefore, is required. Fig. 3 (*f*) shows the arrangement and bending of bent bars giving the maximum spacing when utilizing all the concrete under the bars in diagonal compression, the slope of the diagonal compression being assumed to be 45 degrees. From the geometry of Fig. 3 (*f*), this spacing is seen to be $1.707\,j\,d$, and since from Equation (10):

$$P = 0.707\,\frac{V'\,s}{j\,d} = \frac{V'\,0.707 \times 1.707\,j\,d}{j\,d} = 1.207\,V' \quad\text{.........(20)}$$

the unit diagonal compression is seen to be (see Equation (19)):

$$\frac{1.207\,V'}{1.207\,b\,j\,d} = \frac{V'}{b\,j\,d} = v \quad\text{.....................(21)}$$

For any closer spacing the unit diagonal compression will remain equal to v.

With this arrangement of steel, however, and with the truss conditions assumed, if the tension in the top and bottom horizontal parts of the bar is T, then, in the inclined part, the tension would be $1.414\,T$. Obviously, if the steel stress in the inclined part of the bar is not to exceed that in the horizontal parts, and since P is directly proportional to s, the spacing must be decreased to:

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* See, also, *Technologic Paper No. 314*, pp. 392 and 429.

$$s = \frac{1}{1.414} \times 1.707\ j\ d = 1.207\ j\ d \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

This is the maximum allowable spacing of bars bent at 45° when it is desired to utilize both web steel and concrete to their utmost capacity in resisting shear. Sharp bends are taboo. As before stated, the intensity of the diagonal compression will remain equal to v.

At first glance it may seem impossible that, in an actual beam, a tension as great as T will exist in either the sloping part of the bar or in the upper horizontal portion. Under a moderate load, in a beam reinforced with bars small in comparison with the size of the beam, and which are bent to the largest possible radius, the measured deformation is greatest at the lower horizontal section of the bar, becoming less around the bend and up along the sloping part. As the load is increased, the stress in the sloping part increases more rapidly, and stress is soon found in the upper horizontal part. Soon after the stress in the lower part reaches the elastic limit of the steel, the deformation in the sloping and upper horizontal sections approaches that in the lower horizontal portion. This condition of approaching equality continues until the end, and is especially noticeable when high elastic limit steel is used. Under such circumstances bond, as a dependable factor in design, has been virtually destroyed. In the consideration of proper working stresses the conditions after the elastic limit is passed should always be taken into account. From this, it is evident that in order to make certain of security against diagonal tension failure, particularly when high "shearing" stresses and high tension values are permitted in high elastic limit steel, the ends of all bars should be 100% anchored.

When vertical stirrups are used, without being anchored at the top, the spacing should not be greater than $\frac{2}{3}\ j\ d$, as is clearly shown by the geometry of Fig. 3 (g), and their value may be limited to a less value than this by bond either on the upper part of the stirrup or on the horizontal reinforcement. Even if cracks extend no higher than $\frac{1}{3}\ j\ d$ from the bottom, only the upper two-thirds of a stirrup may be counted in its development; and, since the stress transferred from the stirrup to the concrete in the distance, $A\ B$, is the same as that transferred from the concrete to the horizontal steel in the equal distance, $C\ D$, it is evident that if u has reached its allowable limit on the horizontal bars, Σo of the stirrups above the crack must equal Σo of the horizontal bars, or else the spacing should be reduced in the ratio of these total perimeters. This arrangement is usually difficult to secure, which shows the urgency of effective anchorage at the top as well as at the bottom. Conversely, the bond condition on the horizontal steel is seen to be of vital concern, for it is evident, again, that, since the stress in the stirrup is proportional to the shear, and also since the stirrup stress is limited to the amount that can be transmitted by bond to the horizontal steel in the distance, s, the capacity of the beam to resist shear is also limited. Therefore, wherever stirrups are depended on, firm end anchorage of the horizontal steel is essential to safety. (See, also, Fig. 4.)*

The commonly specified maximum spacings of $\frac{3}{4}$ d for bent bars and $\frac{1}{2}$ d for stirrups were determined from tests conducted years ago on beams. In these tests the stirrups were not anchored at the top, and bent bars had sharp bonds and rarely had end anchors. Because the conditions in the test beams reinforced with stirrups are similar to those shown in Fig. 3 (g), the correspondence of the stirrup spacing deduced by Equation (22) with that determined by tests, is close. If each stirrup of Fig. 3 (g) were effectively anchored at the top and all the diagonal compression could be carried down along a single narrow line to the point where the next stirrup joins the main reinforcement, the spacing might conceivably be increased from $\frac{3}{4}$ d to d. Unlike a steel truss, in which the diagonal compression may be brought down a comparatively narrow member to a pin panel-point, the concrete cannot transfer, at a single point at the junction of the main steel and stirrup, the diagonal compression which it carries. Thus, the advisability of using the closer spacing is evident. Fig. 3 (g) shows also why the first stirrup should not be farther from the support than $\frac{1}{2}$ s.

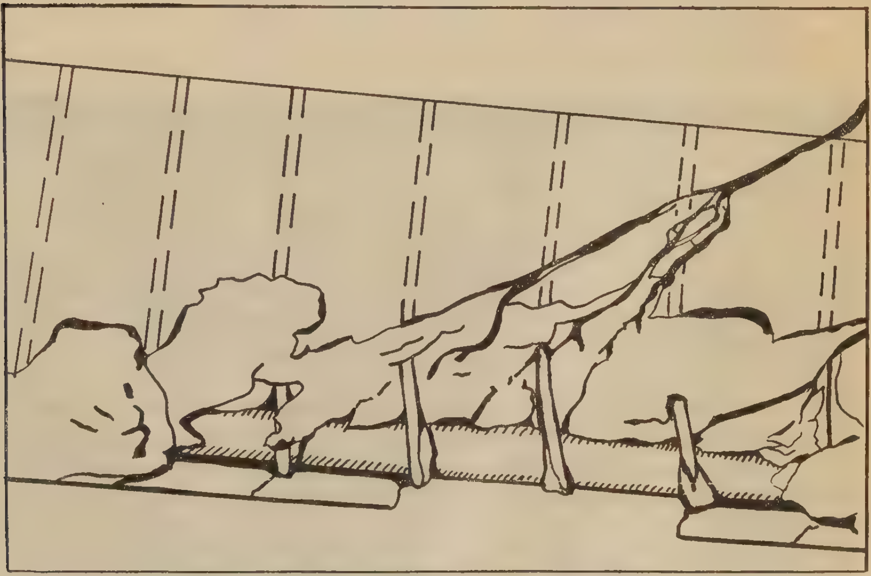

FIG. 4.

The discrepancy between the usual maximum spacing for bent bars, $\frac{3}{4}$ d, with that deduced by Equation (22), is undoubtedly due to the slipping of bars in the test specimens or the crushing and splitting of the concrete beneath the sharp bends.

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* *Deutscher Ausschuss für Eisenbeton,* Heft 20, and *Journal,* Western Soc. of Engrs., January, 1926, p. 11.

Effect of Bond Failure on Shear Failure

The break-down of bond is immediately reflected in a modification of shear conditions. It is common practice to design beams with reinforcement similar to that of Fig. 5. The shear formulas may indicate that the spacing is quite satisfactory, but suppose that the diagonal bars are 1⅜ or 1¼ in. square and are stressed to 16 000 lb. per sq. in. Unless the beam is very deep, there is no possible chance of developing a stress of this magnitude in the sloping part of the bar by bond alone. Hence, practically all the stress is either carried by bearing on the concrete under the bends, or the beam fails at these points.* In a continuous beam, the case is even more serious, for each set of bars crossing the support must be completely developed by bond or bend beyond the point where the previous set was bent down. This bond length is usually short because the bars are bent down at frequent intervals to resist shear; hence, the dependence is largely upon the bend. Fig. 6 shows the result. Even worse, it is the practice in some instances to reduce arbitrarily the computed negative moments over supports.

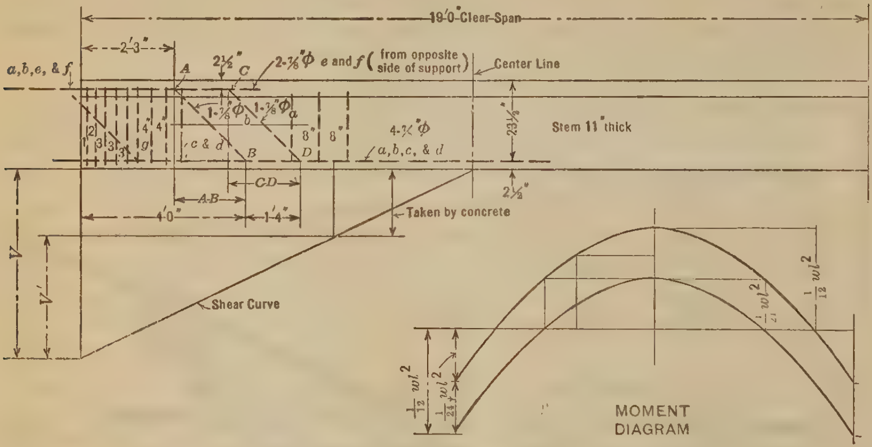
19′0″ Clear Span
2′3″
a, b, e, & f
A
2½″
C
2-⅞″ϕ e and f (from opposite side of support)
Center Line
1-⅜″ϕ
1-⅜″ϕ
b
a
4″
4″
2
1
3
3
3
g
c & d
B
D
8″
8″
4-⅞″ϕ
a, b, c, & d
23⅛″
Stem 11″ thick
2½″
CD
A-B
4′0″
1′4″
Taken by concrete
V
V′
Shear Curve
$-\frac{1}{12}wl^2$
$-\frac{1}{24}wl^2$
$+\frac{1}{12}wl^2$
$-\frac{1}{24}wl^2$
MOMENT DIAGRAM

Fig. 5.—The Effect of Bond Failure on Shear Failure.

The maximum radius of bend that can be obtained is none too great if the "one hoss-shay" beam is the objective. Even then it is wise not to allow the bar to remain in one vertical plane. Crossing bars over to the opposite side of the beam would help to prevent the reinforcement from splitting the beam into laminæ.

Amount of Web Reinforcement Required

Referring again to Fig. 3 (g), it will be seen that the pull on one stirrup is equal to the increment in total tension in the horizontal steel in the distance, s. Neglecting the small amount of shear "taken by the concrete", the total pull on all stirrups will equal the sum of all the increments in the horizontal steel, that is, the total tension. If the same unit stresses are used, the

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* *Deutscher Ausschuss für Eisenbeton*, Heft 12, by Dr. C. Bach and Prof. O. Graf.

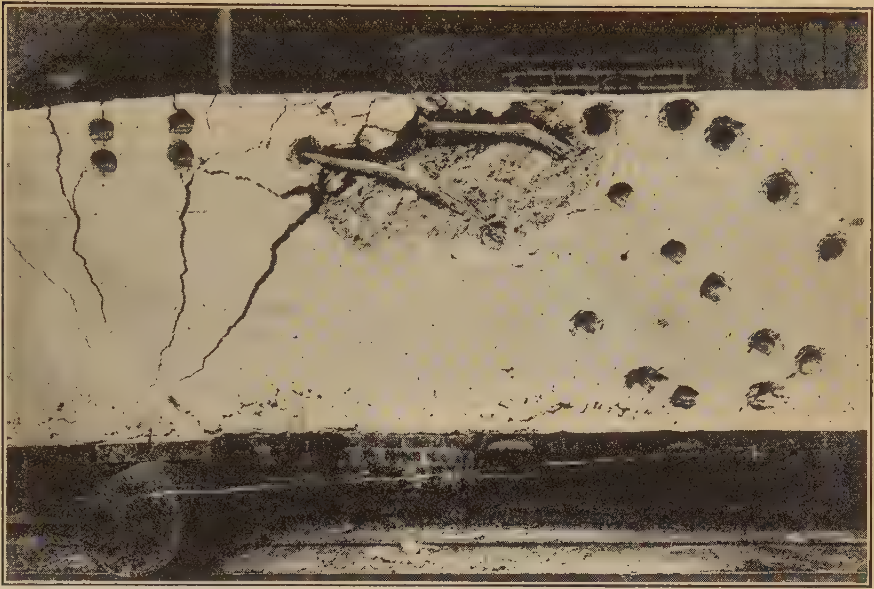

FIG. 6.—SIDE VIEW OF TEST BEAM.

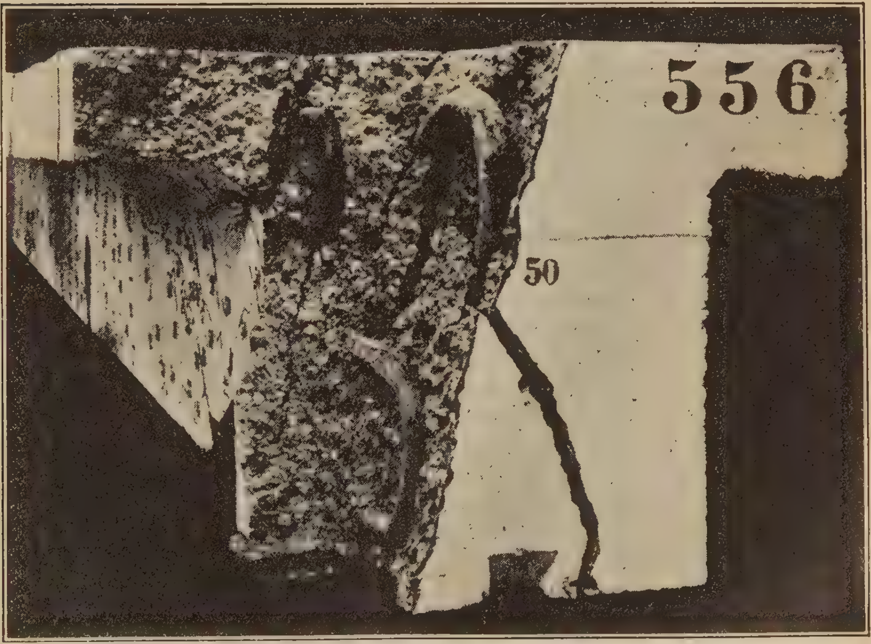

FIG. 7.—END VIEW OF A TEST T-BEAM.

total cross-sectional area of all the stirrups in a simple beam between the point of maximum moment and either support will equal the cross-sectional area of the horizontal steel at the point of maximum moment. In a continuous beam it will equal the sum of the steel areas theoretically required at the point of maximum positive moment and that simultaneously required for negative moment at the support under consideration. If either of these moments is arbitrarily increased to provide for possible inequalities in loading (as is usual), it will, of course, not be necessary to increase the stirrup area. On the other hand, if a lower steel stress is used in stirrups that are arranged alone or in combination with bent bars, the cross-sectional area must be proportionately greater. If the concrete is assumed to take any part of the shear, V, in the sense previously explained, then the total area of the stirrups will be decreased in the ratio, $\frac{V'}{V}$.

Similarly, if bars are bent up when not required for moment, at a spacing not exceeding 1.207 $j\,d$, the tension in the bent part of the bar, at the higher loads from which working stresses are deduced, equals the increment in horizontal steel stress, and again the required area in bent bars equals the cross-section of longitudinal reinforcement.

JUNCTION OF FLANGE AND STEM

In T-beams, the junction of flange and stem is a region subjected to various stresses, which together produce a critical condition (see Fig. 7). There is the longitudinal shear which tends to push the flange lengthwise past the ends of the stem, as well as to produce diagonal cracking of the flange. (See Fig. 8, which is the top view of the flange of a test T-beam.)* There is the tendency for the beam to crack longitudinally as a result of the transverse moment due to the downward flexure of the flanges. There is also a moment resulting from the compression in the outstanding flanges. Let Fig. 9 (c) represent the top view of a T-beam. The center of compression in the outstanding flange, acting at some distance away from the face of the stem, produces a moment, M. Under even moderate loads this will result in a crack, as at d, which rapidly develops from the point of maximum moment toward the end of the beam. If the horizontal shear between flange and stem is high, the failure between the end of the crack and the end of the beam will be sudden. Any load on the flanges, as when the flange is part of the floor system, tends to accelerate such action.

Again, the upper ends of bent bars, as usually bent, are very destructive in this region. Fig. 7,† giving the end view of a test T-beam, illustrates how destructive this action may be. Cracks usually appear at the middle of the upper surface or directly over the bent bars and cut through the beam to the root of the angle between the flange and the stem and under high stress the flange will be thrown off violently.†

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* *Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens,* Heft 122-123, by Dr. C. Bach and Prof. O. Graf.

† See, also, *Deutscher Ausschuss für Eisenbeton,* Heft 12, by Dr. C. Bach and Prof. O. Graf.

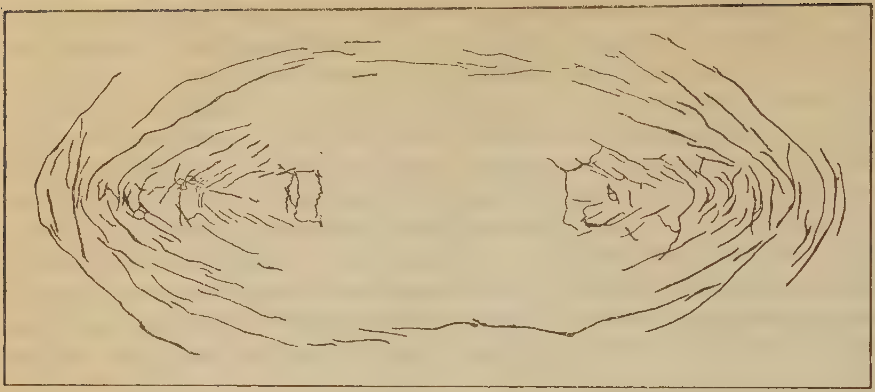

FIG. 8.—TOP VIEW OF THE FLANGE OF A TEST T-BEAM.

Clearly, cross-reinforcement is essential. As already implied, the amount should be at least equal to the cross-sectional area of the main longitudinal steel, since reliance on the concrete at such a place is unsafe. However, it is evident that since the T is called upon to resist positive moment only, in continuous T-beams the area of cross-reinforcement needs only to equal the area

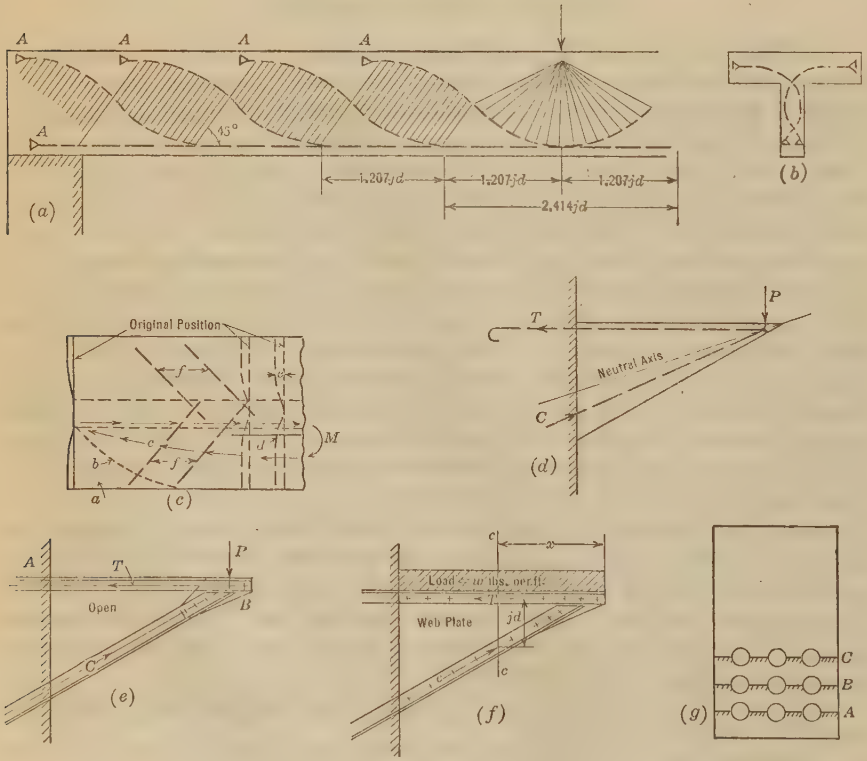
A
A
A
A
A
45°
1.207jd
1.207jd
1.207jd
2.414jd
(a)
(b)
Original Position
f
c
d
M
c
b
f
a
(c)
T
P
Neutral Axis
C
(d)
A
T
P
Open
B
C
(e)
c
x
Load w lbs. per ft.
T
Web Plate
jd
c
C
(f)
C
B
A
(g)

FIG. 9.—MISCELLANEOUS DETAILS.

of the longitudinal steel-resisting positive moment. The disposition of this reinforcement is of considerable importance. If placed in the form of cross-bars only it can effectively resist the transverse flexure caused by load on the flanges, as well as the moment, *M,* of Fig. 9 (*c*), and the splitting tendency of hooks on the main reinforcement. It cannot resist the tendency to push the slab lengthwise on the stem, nor the formation of the diagonal cracks in wide-flange T-beams. (See Fig. 8.) Such forces will merely bend the stirrups out of line as shown at *e,* Fig. 9 (*c*). All the forces may be resisted, however, by steel placed at an angle of 45° with the axis of the beam, and effectively anchored at both ends, as at *f,* Fig. 9 (*c*). This is perhaps most easily effected by bending all the main reinforcement up through the stem and out into the flanges, anchoring the ends firmly. (See Fig. 9 (*b*).)

If the main reinforcement is not bent up and outward into the flanges the stress in the cross-reinforcement should be kept low, say, not more than 10 000 lb. per sq. in., in order to prevent incipient splitting and damage from hooks. The required area of steel used will thus be proportionately increased.

The amount of cross steel in the flange is often expressed as a percentage of the flange area, but it would seem more nearly direct to specify it in terms of the longitudinal steel.*

Wedge-Shaped Beams

Just as with the bond formula,† the usual formulas for shear do not apply to beams in which the main tension reinforcement is not parallel to the compression face. In Fig. 9 (*d*), for example, both the tension, *T,* and the compression, *C,* are constant from load to support; and, from statics, the vertical component of *C* must be equal to the load, *P.* If the beam is proportioned so that the concrete is not over-stressed in compression, then shear failure cannot result, and no shear reinforcement is necessary. In such a beam the critical condition arises from the total lack of bond and the complete dependence on anchorage of a kind that will not damage the concrete.

This case is analogous to that of the structural steel bracket shown in Fig. 9 (*e*), and no designer will question the necessity of complete development at *A* and *B,* nor will he worry about shear in the diagonal strut as long as it is good in compression.

If the load is uniformly distributed, as in Fig. 9 (*f*), then either the top member must be capable of resisting bending, with its accompanying shearing stresses, or a web-plate must be introduced to take the net shear. The vertical component of the inclined compression on any vertical section, *C-C,* is no longer equal to the shear on the section. In Fig. 9 (*f*), the horizontal component of the inclined compression, at Section *C-C,* is:

$$c_h = \frac{w\,x^2}{2\,j\,d} \quad \text{.............................} (23)$$

The vertical component of *c* is:

$$c_v = \frac{c_h\,j\,d}{x} = \frac{w\,x^2}{2\,j\,d} \times \frac{j\,d}{x} = \frac{w\,x}{2} \quad \text{................} (24)$$

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* See, also, *Deutscher Ausschuss für Eisenbeton,* Heft 12, by Dr. C. Bach and Prof. O. Graf.

† See, *Journal,* Western Soc. of Engrs., January, 1926.

That is, in a beam of triangular elevation, uniformly loaded, in which the flanges alone are assumed to resist the tension and compression due to bending, the vertical component of the inclined compression or tension at any section is equal to one-half the vertical shear on that section. The web must resist the other half. Therefore, if a uniform load instead of a concentrated load were applied to the reinforced concrete beam of Fig. 9 (*d*), the web reinforcement would have to be capable of resisting one-half the vertical shear at any section.

If the beam does not come to a point, but has any one of the forms shown in Fig. 10, then analysis* will show that in the general case the shear to be resisted by the web reinforcement is equal to the total shear on the section minus the vertical component of the inclined main stress. This result is confirmed by an inspection of the simple statics of the case, and would apply not only to sloped-top footings, counterforts, balcony girders, and crane girder brackets, but also to the brackets introduced at the supports of continuous beams. The latter, if not sloped too sharply, will assist, therefore, in resisting both compression and shear.

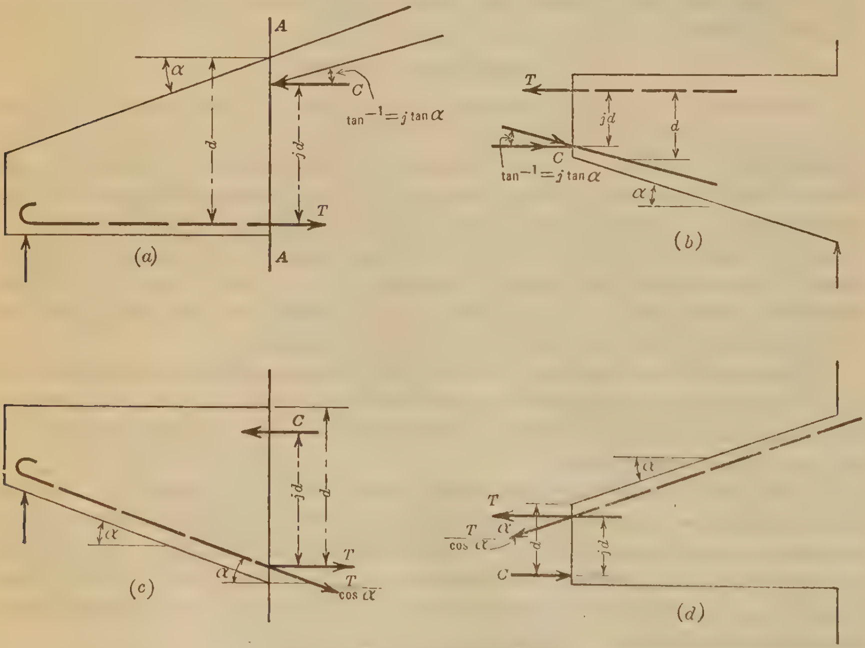

FIG. 10.—ODD-SHAPED BEAMS.

CROSS-SPACING OF THE MAIN REINFORCEMENT

The spacing of the main reinforcement with reference to the cross-section of the beam is wholly or partly dependent on the value of the concrete in pure shear. Let Fig. 9 (*g*) represent the cross-section of a simple beam. If none of the bars is bent up, any difference in stress on the bars, *A*, between two adjacent vertical cross-sections must be taken up by bond on the surface of the bars and transferred upward by horizontal shear to the compression side of the beam. That part of the bond on the lower half of the bars must be transferred across the narrow section of concrete at Section A. If the value of the concrete in horizontal shear is the same as the intensity of the allowable bond, then the clear space between the bottom bars should not be less than $\frac{1}{2}\pi d$. Similarly, the spacing of the bars on the next layer should not be less than $\frac{3}{2}\pi d$.

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* Similar to that in *Journal,* Western Soc. of Engrs., January, 1926, pp. 21–22.

Based on this reasoning, specifications often include clauses limiting the bars to two layers and the spacing to three times the diameter. The value of good concrete in shear, however, is four or five times as great as its value in bond. In a simple beam uniformly loaded, too, the bond stress developed is small where the bars are most numerous; that is, at the point of maximum moment; and near the ends, many of the bars have either been bent up or cut off, leaving a greater net horizontal section of concrete. Thus, it is not particularly objectionable to have three or four layers of bars in such beams.

In the continuous beam the same holds true for the part subjected to positive moment, but in the region of maximum negative moment the highest bond stress exists at the place where the negative reinforcing bars are most numerous. Here, conditions should be carefully watched and the effective anchorage of the negative steel is again seen to be of great importance.

A practical feature, often overlooked, has a great deal to do with bar spacing, namely, the size of the coarse aggregate used. If large stones lodge between bars, there will always be open pockets beneath, and cases sometimes occur in which the concrete around the lower steel can easily be removed.

If the bars are bent in such a manner that the bends will serve as anchors, this is more unsightly than serious; but, if the bars are straight, particularly if some do not reach the support, they are of little more use than if simply stuck on the bottom of the beam. It need scarcely be added that to stagger the rows of bars so that each bar comes directly over the space between the bars in the layer below is to invite trouble.

CONCLUSION

The purpose of reinforcement is to make two wholly unlike materials act together as nearly as possible as one homogeneous material. Many of the difficulties have been met with empirical rules, but in a few cases unique ideas of some merit have been tried. In 1909, Locher* arranged the reinforcing bars so as to follow the tension stress trajectories. Later, Gilchrist† designed some heavy beams using the same principle. The writer's suggestions have been advanced with somewhat the same idea.

Many of these suggestions may seem extreme; for example, crossing bars to the opposite side of the beam; bending them with extremely large radii; bending them out into the flange; and providing them with anchorage capable of developing the ultimate strength of the steel without appreciable slip. No form of hook will accomplish the last of these. With steel of structural or intermediate grade some of these precautions might be unnecessary, for due to the comparatively early yield of the steel, failure would also be comparatively early. The yielding of the steel would cause undue deflection, which, in turn, would concentrate compression stress near the extreme fiber and result in "compression failure". In such a case anchorage would be of use only in developing the yield point of ordinary steel. On the other hand, bond failure is responsible for failure in many beams in which the steel stress is less than the yield point. Many failures of this type have been called "tension failures" or "diagonal tension failures". It is surprising to what an extent the strength of a beam may be raised, without changing the quality of the concrete, by the use of steel with a high elastic limit, designed so as to eliminate the numerous types of secondary failure discussed. It may be added that all the precautions listed are necessary in order to achieve this object. A brief résumé of these suggestions will show their bearing on design.

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* See Marsh, "Reinforced Concrete," Third Edition, Second Imprint, 1909, p. 24.

† *Engineering News,* March 12, 19, and 26, 1914.

Let Fig. 9 (*a*) and Fig. 9 (*b*) represent a simple beam carrying a single concentrated load at mid-span. The steel should have a high elastic limit. The number of bars chosen should be such that they can be bent up at a spacing not exceeding 1.207 $j\,d$. The bends should be of the largest possible radius. This is accomplished by bending the bars so that at mid-height they would have a slope of 45°, bending horizontal in the top again at the same radius. The ends should be fixed with anchors that cannot damage the concrete. If the beam is T-shaped, the alternate bars should be bent out into opposite flanges. For the sake of symmetry, it would be preferable to bend up the bars in pairs or in groups of four, one bar, or pair of bars, from each group being bent outward into each flange. The last bent bar, or pair of bent bars, should become horizontal over the support, leaving one horizontal in the bottom of the beam. Thus, practically all the steel transfers its own stress to the compression side of the beam, eliminating questions of diagonal tension and connection of flange to stem.

With such an arrangement of reinforcement—bars being bent up gradually where not needed for moment—the effective cross-section of the bars varies from a maximum under the load to nearly zero at the support, which will result in a practically constant unit steel stress from mid-span to the support. The steel stress being constant, bond stress plays no part in the design, but full anchorage is essential. In T-beams it is advisable to bend the bars from one flange over to the opposite side of the stem, as shown in the end view of Fig. 9 (*b*), to preclude any possibility of the bars pulling through the corners. This will also have the effect of helping to prevent the splitting of the beam into laminæ. The structure thus becomes equally strong in all parts. Such an arrangement was devised a few years ago by William Fry Scott, M. Am. Soc. C. E.—the tests being witnessed by the writer—and the results confirm in every case the statements of this paper.

With these suggestions the designer can extend the application of the principles mentioned to the cases of beams uniformly, or otherwise, loaded, continuous beams, counterforted walls, and other structures.

DISCUSSION

Edward Godfrey,* M. Am. Soc. C. E. (by letter).—The writer is in complete accord with Professor Mylrea in his emphasis on the importance of re-examining "the basis of structural theory to check the assumptions on which it is founded." This is particularly true in any case in which the basic theory has been questioned and the arguments of the questioner have not been squarely met and answered.

One of the most vital questions in the field of reinforced concrete design is the stirrup or short shear member. It is of exceeding importance that free discussion be entered into and a conclusion reached as to the value of the stirrup—if it has any real value—and its proper place in design.

Professor Mylrea clearly considers of no importance the fact that stirrups in a whole beam have little or no stress. In the writer's judgment there is a real significance in this fact of test and analysis which is closely connected with the efficiency of stirrups as shear members. Experimental evidence shows that stirrups do not act until the concrete cracks. Analysis anticipates the same thing, for there can be no action in a homogeneous concrete beam that would elongate the beam in a vertical direction. Conceding the fact, then, that the stirrups in a reinforced concrete beam do not come into action until the beam cracks, the question arises as to what this involves. The unit shear allowed by some modern standards on the cross-section of a beam is a large fraction of the shear at which the first crack in the web takes place in test beams.

Modern specifications allow a large share of the beam shear to be taken by the concrete. The margin between the shear allowed on the concrete web and that which cracks the concrete is small. In fact, it is not unusual to find beams in which the concrete web is cracked under safe load. What of the part of the shear allotted to the concrete web? A cracked web, unaided, could take no part of this shear. It will be thrown, therefore, into the stirrups, and it is largely in excess of the shear for which the stirrups are designed. Here is a condition, then, that can be expected in a standard design, namely, large over-stress in the stirrups.

In no other analysis of stress action is it assumed that any part of a structure is idle and inoperative until some other part fails. The writer does not believe this is sound analysis, particularly since there is a means of reinforcing a beam for shear that is free from this objection, both in its analysis and in its agreement with tests.

Professor Mylrea refers to tests by A. N. Talbot, Past-President, Am. Soc. C. E., which proved that stirrups take no stress until the concrete cracks in shear. Tests by this same experimenter† prove that beams reinforced for shear by bending up the main reinforcing rods with a long flat bend, gave high stresses in the bent-up part of the rods. Furthermore, in the beams reinforced with stirrups, in an actual building, cracks were easily found in the concrete; whereas in beams reinforced for shear by bending up rods with a flat angle and crossing them over the support, no cracks could be found.

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* Structural Engr., Pittsburgh, Pa.

† *Bulletin No. 64*, Univ. of Illinois Eng. Experiment Station.

Analysis agrees perfectly with this, for both the shear deflection and the deflection due to bottom tension are accompanied by elongation in the beam in the diagonal direction in which the bent-up and anchored rods lie; and, since the rod is there to take the stress, cracks in the concrete are prevented.

A simple test and one that can be made in an office is as follows: Take a straight stick of wood about as long as a desk and about 1½ by 2 in. in cross-section. Fasten mandolin wires in screw-eyes as indicated in Fig. 11. Introduce small bridges to give a tension. Place this beam on two books and pick the wire. Then press down at the middle of the stick. The tone of the wire will immediately go up, proving the existence of tension in the direction in which the wire is stretched. It is in this position and direction that bent-up main rods are or should be placed, and every increment of load adds stress in these rods. Furthermore, failure of the beam in shear is impossible unless this rod severs or is pulled out of its anchorage. This cannot be said of any stirrup installation.

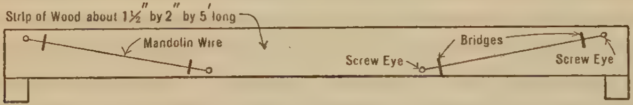

FIG. 11.—EXPERIMENT TO DETECT TENSION IN BENT-UP ROD.

It is manifest that a test made on this wooden beam with vertical wires on the sides, corresponding to stirrups, would not exhibit any increment of tension in the wires. Sharply inclined wires would show little if any added tension.

The author states that the introduction of vertical stirrups would greatly delay failure, and this, he points out, is the function of such stirrups. Unless they can prevent failure entirely, and surely, some other form of shear reinforcement is a necessity. He assumes that the concrete beam cracks at an angle of 45° with the vertical, and the stress in a stirrup is worked out on this assumption. The angle of this crack is the determining factor in finding the stress to be allotted to the stirrup, so there should be some good reason for the selection of 45 degrees. If the tangent of the angle is more than one, the stress in the stirrup is less in the inverse ratio. If that tangent is less than one, the stress allotted to the stirrup is greater.

There is no law that would compel the cracks to take an angle of 45 degrees. In fact, Fig. 6 shows cracks that are almost vertical. The writer has seen many actual beams and has read descriptions of many more that failed in wrecks by cracking at the line of support in practically a vertical plane. When actual beams fail at wall supports by breaking part of the concrete over the support itself, and when they carry with them the stirrups that were intended to prevent this failure, as the writer has observed to be the case, what ground can there be for the assumption that the concrete will crack some distance from the support and at an angle of 45° with the vertical? In Fig. 6, the cracks start at the support.

Continuous and restrained beams tested by Withey* are shown in many cases to have cracked exactly at the support. These beams had stirrups, but were not reinforced by bending and anchoring the main reinforcing rods as the writer would have done.

Assume a case in which a 45° line would cross three stirrups, and suppose, instead of cracking at 45°, the crack crossed only one stirrup. The stress in this stirrup would be three times that allotted to it by the designer. This increase of 200% in the stress in the stirrup would be merely the result of a crack in a concrete beam taking a different course from that assumed by the designer, and there is nothing inherent in the beam that would prevent it from cracking in that manner. In fact, the crack may miss the stirrups altogether, and a beam designed according to standard rules could fail in shear without in any manner disturbing or stressing the steel reinforcement placed in that beam to take the shear.

It is a fact that, between any two stirrups and between the last stirrup and the support, there is nothing to take the shear of a beam except the concrete and the horizontal steel. These would be there if no stirrups were used.

The author uses the truss analogy to explain and determine the stress in the stirrup. In the truss analysis the web members cut by any section, whether vertical or inclined at any angle, must be capable of taking all the shear of the truss. The analogy breaks down, therefore, when the inclination of the cut is arbitrarily assumed to be 45 degrees. The case then resolves itself into one of an arbitrary allotment of stress having no relation to steel truss analysis.

In another vital respect the truss analogy fails. In every truss, in steel design for example, every pound of stress in a web member must be traced through some adequate detail to an adequate resistance and, finally, to the reaction of the support. Professor Mylrea's analysis does not follow the stress of the stirrup beyond the end of the stirrup itself. The stirrup is said to be anchored. The nature of this anchorage corresponds to the end detail of a truss member. It should, of course, be capable of taking the full stress of the stirrup. The paper does not place much emphasis on the need of adequate anchorage, or of ways in which it can be obtained, and this should be a matter of searching inquiry, since it is a governing feature in the strength of the beam.

The anchorage at the bottom of the beam seems to be by a loop around one or more reinforcing rods. At the top of the beam it appears to be a short hook around a reinforcing rod or merely in the concrete. These are the standard and common end anchorages for stirrups.

No designer would seriously consider that a rod ½ in. square, for example, is adequately anchored for its safe load as a hanger if it is merely buried with a small hook about 2 in. long, or even looped about a rod close to the bottom of a beam with this same hook. Why should not the same principles apply in the case of a stirrup as would obtain in the case of a hanger on which an actual load is to be applied?

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* *Bulletin No. 115,* Univ. of Wisconsin Eng. Experiment Station.

If it is assumed that the concrete itself furnishes the anchorage for the stirrup, how does that concrete deliver the shear load to the support where it must ultimately go? In a whole beam how does this avoid or lessen the diagonal tension in the concrete web? And if it does not accomplish this, why use the stirrup? In a beam, cracked at an angle of 45°, how can the concrete at the tip of the triangle in Fig. 1 (*h*) be considered capable of anchoring the last stirrup cut by the 45° plane to the full value of that stirrup? Here, again, a designer would not seriously consider placing a capacity load on a hanger anchored in this manner.

By Professor Mylrea's reference to Fig. 2(*g*), it is evident that he considers the pull of a stirrup to go directly into the horizontal rods as a vertical load. This means that the load of the beam is hung as a vertical shear load on the horizontal rods. Analysis cannot justify this disposition of the shear load. Steel rods embedded in concrete cannot take any appreciable shear until the concrete fails in shear. The idea that steel rods as pegs could take shear at 10 000 to 12 000 lb. per sq. in. was discarded from specifications long ago. After the concrete fails in shear the horizontal rod would be in bending, and the capacity of a reinforced concrete beam would be little if dependence were placed on the bending strength of the rods to carry the shear to the supports.

While the author emphasizes the necessity of anchoring the web reinforcement "at both top and bottom strongly enough to develop its full tensile strength", he does not explain the nature of that anchorage, nor how it could be effected at the top and bottom of the beam, which mark the extreme ends of the stirrups. *Technologic Paper No. 314,* U. S. Bureau of Standards, is referred to several times. This report describes tests in which the stirrups on shear members were fully anchored at the level of the top and bottom of the concrete web. In fact, they were anchored into heavy beam heads which, in the investigation, were eliminated from consideration, so far as shear is concerned. For this reason the writer believes that those tests were of little or no value as teaching any lesson concerning reinforced concrete beams. Such beams, as constructed, do not carry with them large surplus depth that can be ignored in the calculation and yet be taken advantage of as anchorage space for stirrups.

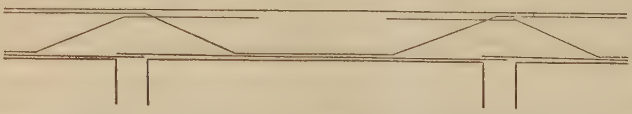

FIG. 12.—MAIN BARS ARRANGED TO RESIST SHEAR.

Beam analysis must be based on beams as it is practicable to construct them. The tests described by the U. S. Bureau of Standards were on a comparatively thin concrete web fixed in a heavy concrete frame into which the web members were fully anchored and which was quite outside the area considered as concrete web. This does not correspond in any way with a reinforced concrete beam; hence analysis of a reinforced concrete beam cannot properly be based on results of these tests.

The author emphasizes the importance of properly anchoring stirrups "at the ends". Also, he states that "sharp bends are taboo". He does not refer to the impossibility of fully anchoring a rod at its end. Unless a nut and steel plate is used, full anchorage at the end of a rod is not possible. A small hook either in concrete or around a main rod is taboo, because it involves a sharp bend. Anchorage in concrete requires a considerable part of the entire rod. Professor Mylrea states that no form of hook will accomplish the anchoring of steel to develop the ultimate strength without appreciable slip. It would not be an unusual condition for a stirrup to require one-half the depth of the beam for the 100% anchorage that Professor Mylrea states is necessary. The web member is of zero length, and the truss analogy fails.

The author rightly emphasizes the importance of end anchorage of main reinforcing rods. In fact, with the top and bottom rods firmly anchored into the support, a large shear can be carried by the concrete even if it is cracked. The friction acting between the compression surfaces—if the crack is vertical—will take the shear component. It is allocating this shearing strength of the concrete web to stirrups, in interpretation of tests, that has given the stirrup a place in design that its nature does not warrant.

If a continuous beam should crack a short distance from the support, namely, at the point of inflection, there would be no compression surface, because the moment is zero. A vertical crack at this location would create the dangerous condition of all the shear being taken by horizontal rods. If main reinforcing rods are bent up and crossed over the supports, as in Fig. 12, any section, cut at any angle, between the support and the bend in the rod, has a positive shear reinforcement that effectively prevents failure by direct tension in that reinforcement. The main rods are amply capable of "carrying" this tension and "delivering it" to its anchorage over the support where the reaction of the beam must be met.

Professor Mylrea intimates that in future designs higher unit stresses will be used in concrete webs. The tests of the U. S. Bureau of Standards, to which he refers, showed cracks in the concrete webs at units of 200 to 300 lb. per sq. in. A concrete section nearly twice the nominal web area was actually in shear. These test beams, which were reinforced with stirrups with an efficiency that could not be approached in a practical design, failed in their concrete webs at about the allowed safe unit shear of standard specifications. They, therefore, did not even justify the units of those specifications.

C. A. P. Turner,* M. Am. Soc. C. E. (by letter).—The author attempts to analyze the web resistance of concrete beams which is a special case of the general problem.

Prismatic beams of homogeneous material were treated by Mariotte on the assumption ("on peut concervoir") that there were as many fibers compressed as stretched and, later, because the cross-section of the prism resisting horizontal shear was far greater than that resisting moment stresses, horizontal shear strain was neglected in the Bernoulli-Eulerian investigations which constitute the common theory of flexure as taught to-day. Mathematically considered, this neglect of shear strain is equivalent to treating the shearing modulus of the prism as infinite. This is the same as stating that a plane section normal to the axis remains normal to it after flexure and without distortion; that is, it still remains plane. This important normal relation is commonly overlooked in stating the hypotheses of the ordinary theory. An exact analysis of the trapezoidal components of deformation of the prism was thus developed, needing only the superposition thereon of the pure shear distortions to arrive at the exact theory of beams. The common theory is sufficiently approximate for the practical computation of the elastic behavior of simple prismatic beams having a length ten times their depth, but when the horizontal cross-section is reduced to a thin web, as in the broad flanged I-beam, or in the case of reinforced concrete beams, shear strain cannot be neglected for even approximate accuracy. For the restrained beam the greatest tensile fiber stress computed by the common theory is ordinarily from 20 to 30% greater than when computed by exact theory. Deflection in restrained beams is reduced by the interference of shear distortion. In the simple beam, on the contrary, the deflection is greater than that computed by the ordinary theory but the extreme fiber stress is the same at the section of maximum moment for symmetrical loading.

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* Cons. Engr., Minneapolis, Minn.

Weisbach suggested* the influence of pure horizontal shear on the position of the neutral plane in the homogeneous beam as one of the problems in developing the exact theory of flexure, but he neglected to solve the problem. Professor Karl Pearson proved that a neutral plane (defined as the locus of points at which tension and compression balance) does not in fact exist along the length at the gravity axis of the simple homogeneous beam. The writer has found† that the true neutral plane of the homogeneous beam curves downward toward the end of the beam approximately nine-tenths of the depth by equilibrating—by the parallelogram of forces—the center of the shear stress area as the reaction to the fiber stress area.

The true location of the neutral plane is thus a curved surface of the same order as the slope curve. This is true also of the neutral plane of the reinforced concrete beam. The stress mechanism in the composite beam differs, however, from that in the homogeneous beam in that, although at the neutral plane there are right shearing stresses, they are oriented differently. The curves of horizontal shear are tangent to the neutral plane in the homogeneous beam, but they intersect the neutral plane of the composite reinforced concrete beam because the shear between the concrete and metal is equilibrated at the surface of the steel at the bottom of the beam. Accordingly, instead of curves of principal tension, curves of horizontal shear pair (symmetrically opposite, that is, with respect to the neutral plane) with curves of principal compression and the curves of tension in the concrete cut across the beam, assuming a position in the tension zone comparable to the curve of greatest shear in the homogeneous beam. With such a stress mechanism it is obvious that the neutral plane (in which the intensity of tension, compression, and shear are equal) is compressed along its length in proportion to the load applied as shown, but not recognized, in tests by the U. S. Bureau of Standards.

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* "Mechanics of Engineering," 1846.

† "Elasticity and Strength of Materials," Section II.

Accordingly, the neutral plane in the composite beam cannot be located by finding the locus of zero stress as may be done with a homogeneous beam. This radical difference in stress mechanism between the homogeneous and composite beam renders the investigation of the characteristics of the concrete beam by analogy to the homogeneous beam somewhat misleading. In vertical planes parallel to the axis, however, the curves of displacement are similar to those of the homogeneous beam, intersecting the neutral plane at a more acute angle than is the case with the homogeneous member.

There are two kinds of rotational distortion in a beam whether of reinforced concrete or homogeneous material. The rotational strain that is caused by the sharpness of curvature represented by trapezoidal deformation gives rise to equal diagonal tensions in the face of an originally square element in the tension zone and to equal diagonal compressions in an originally square element in the compression zone. The intensity of these diagonal stresses increases from zero at the neutral plane to a maximum at the outer fiber. This kind of strain causes cracking in the shallow beam toward the center.

The second kind of rotational deformation is that of pure shear, or the tendency of the outer fibers to slide as two planks slide, one upon the other, when supported at their ends and loaded. This deformation produces a rhombic (pure shear) distortion, which is zero where the moment area is divided into equal segments (moment areas to the right and left of this section give rise to opposite shears which are equal to each other as action and reaction are equal) and increasing from that locus toward the end of the beam when simply supported. In rhombic distortion one diagonal of an originally square unstrained element is stretched; the other is compressed. Consequently, the maximum diagonal tension will occur at the location where these deformations, superimposed one upon the other, give the greatest diagonal strain. Because the rhombic deformation is proportional to the slope and the trapezoidal deformation is proportional to the sharpness of curvature, which, in turn, is proportional to the bending moment, it is possible to lay off two curves —one upon the other—so as to predict the location of the first crack that may occur in a concrete beam. The upper curve is of the same order as the slope curve, and the lower curve the same as the moment curve. If the maximum ordinates of these curves are made proportional to the deformation which they represent for a given beam, then the greatest diagonal tension will occur where the joint ordinate is greatest. It is thus possible to determine the location of the first crack in the concrete beam or the place at which the web of an I-beam will buckle for a given length of beam under a given load.

From the equations of beam theory the external shears are equal to the sum of the loads from the center toward the end. By taking the summation of shear areas from the end toward the center of a beam the magnitude of the moment ordinates can be determined. The summation of moment areas, in turn (divided by EI), gives the tangent of the slope angles along the loaded beam, and the summation of these tangents determines the deflection according to the ordinary theory of flexure. Because the horizontal shears are determined by summing the horizontal fiber stresses, rhombic shear strain is the reaction to the moment stresses. Hence, it is proportional thereto and the shear deflection, therefore, bears a constant ratio to the moment deflection in a given elastic beam for any loading whatever; but this ratio differs in beams of different span lengths because the magnitude of the rotational pure shear distortion depends on the relation between the horizontal section resisting horizontal shear and the vertical cross-section resisting moment stresses. The relative magnitude of the shear deflection in beams of various lengths follows a different proportionate law from the magnitude of moment deflections.

As a steel I-beam has the flange resistance concentrated at the top and bottom, its web stresses are comparable to those of a concrete beam in which the tensile flange resistance is concentrated in the reinforcing steel. A graphical analysis of the rotational strains of the web of the steel I-beam will illustrate the solution of both problems.

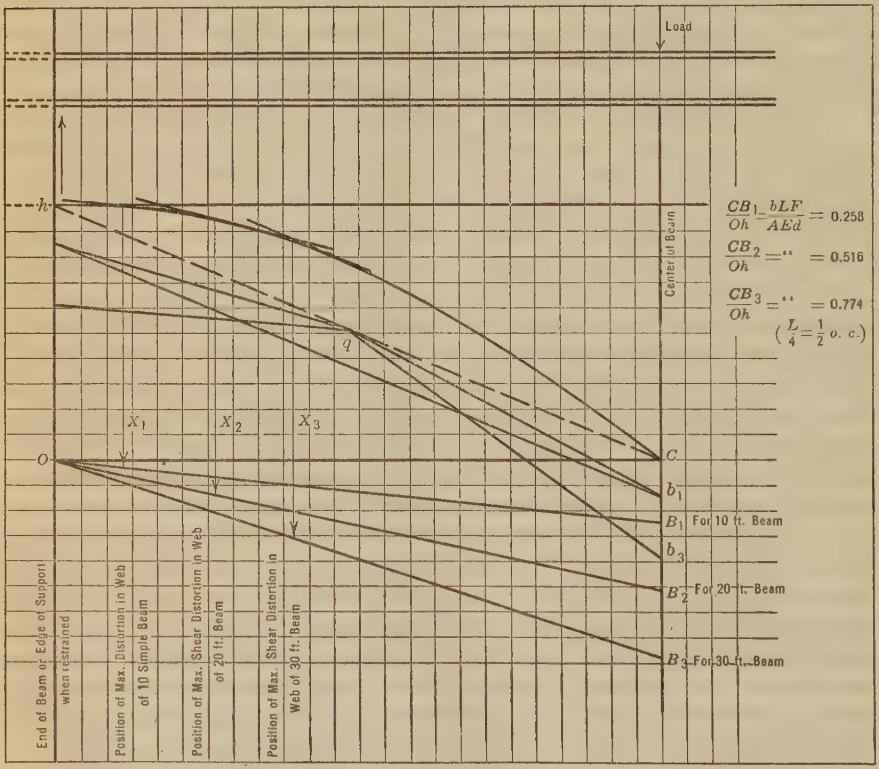
Load
h
O
q
X_1
X_2
X_3
Center of Beam
$\frac{CB_1}{Oh} = \frac{bLF}{AEd} = 0.258$
$\frac{CB_2}{Oh} = \text{''} = 0.516$
$\frac{CB_3}{Oh} = \text{''} = 0.774$
$(\frac{L}{4} = \frac{1}{2}\ o.\ c.)$
C
b_1
B_1 For 10 ft. Beam
b_3
B_2 For 20 ft. Beam
B_3 For 30 ft. Beam
End of Beam or Edge of Support
when restrained
Position of Max. Distortion in Web
of 10 Simple Beam
Position of Max. Shear Distortion in Web
of 20 ft. Beam
Position of Max. Shear Distortion in
Web of 30 ft. Beam

FIG. 13.—ANALYSIS OF ANGULAR SHEAR DISTORTION IN WEB OF 15-INCH, 38-POUND I-BEAM CENTRALLY LOADED, LENGTH SIMPLY SUPPORTED AND RESTRAINED.

In Fig. 13 let *O C* represent the half span with a concentrated load at the center. The moment curves are, therefore, straight lines. The slope curve is of the second degree and the magnitude of the rotational strain represented by *O h* at the end and the ordinates of the curve, *h C*, may be taken as representing the relative magnitude of the pure shear strains. The magnitude of the trapezoidal distortion is proportional to the ordinates measured from *O C* to

$O B_1$ for the 10-ft. beam; $O B_2$ for the 20-ft. beam; and $O B_3$ for the 30-ft. beam.

The sum of the ordinates of the two rotations is a maximum for the 10-ft. beam at X_1; for the 20-ft. beam at X_2; and for the 30-ft. beam at X_3. Conversely, if the beam is restrained at its ends the moment passes through zero at q and the closing line of the slope curve is $h C$. Then $q b_1$ and $q b_3$ represent the moment curves for the center or suspended-span part, and corresponding curves to the left of q are for the cantilever part. Obviously, the maximum rotational strain in the restrained beam is roughly one-third as great as in the simply supported beam of the same section subjected to the same external shear force. This illustrates the illogical nature of the rules of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete* and building code laws in computing the web stress of beams. Fig. 13 discloses clearly why, as the beam becomes longer and more shallow, the maximum rotational strain moves from the end toward the center in the simple beam and from the quarter-point toward a position between the quarter-point and the support, as the span increases in the restrained beam. It thus becomes possible to predict with a high degree of certainty where the first crack in a concrete beam will occur, or where the web of the steel beam will commence to buckle. Because the concrete has a smaller limit of rotational strain before cracking than that possessed by the steel before the yield point is reached, proper attention must be given to the percentage of steel used in the shallow concrete beam.

If the rigidity of the beam is such that plaster will not crack, its deflection under working loads should not exceed $\frac{1}{1\,200}$ of the span.

In order that the deflection shall not increase more rapidly than the fixed proportion of the span length it is customary, with a long shallow beam, to reduce the steel percentage; this requires the development of a rational deflection formula.

For a given fiber stress in the homogeneous beam, or a given steel stress and a fixed percentage of steel in the concrete beam, the deflection increases with the square of the span and it is desirable to develop the relation of the percentage of steel. Turneaure and Maurer have developed the following formula on the assumption that the concrete may resist tension:

$$D = \frac{c_1}{c_2 E_s} \frac{f_s k^2}{d} (p j) \frac{n}{\alpha}$$

This is developed from the ordinary flexure formula by using as the moment of inertia with respect to the neutral axis of the section in compression, $\frac{1}{3} b k^3 d^3$; that of the concrete in the tension zone, $\frac{1}{3} b (1 - k)^3 d^3$; and that of the steel, $nA (1 - k)^2 d^2$. Hence,

$$I = \frac{1}{3} [k^3 + (1 - k)^3 + 3 n p (1 - k)^2] b d^3$$

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* *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, pp. 1193 *et seq.*

or,*

$$\alpha = \frac{1}{3}\left[k^3 + (1-k)^3 + 3\,n\,p\,(1-k)^2\right]$$

In explaining this formula Messrs. Turneaure and Maurer note that the concrete may crack, but apparently they are inclined to consider that it resists tension after it has been cracked by tension. However, the fact that horizontal shear in the concrete beam, parallel with the steel, replaces horizontal tensions of the homogeneous beam furnishes the solution of this enigma.

The manner in which horizontal shears may cause or oppose bending in the concrete beam may be illustrated by testing a 12 by 16-in. beam, 14 ft. long, centrally loaded, but resting on symmetrical supports 8 ft. apart, allowing 3 ft. over-hang at each end. With 2% of reinforcement the first crack in the concrete will occur in the top of the beam in the overhanging part where there is no applied moment; hence, it is apparent that the horizontal shear in the concrete, parallel with the steel, actually forms that part of the bending resistance which has been erroneously attributed by theoretical writers to the tensile resistance of the concrete.

With this explanation the moment of inertia developed by Turneaure and Maurer appears to be satisfactory. It shows clearly that the percentage of steel should be reduced as the span increases, and that the tension zone of the concrete may not be disintegrated by repeated loading if the designer wishes to develop, economically, the value of the steel element.

Stirrups are of much less value than is commonly supposed because of the nature of the stress mechanism, and bent-up rods should follow the curve of tensile displacement which is about 30° to the horizontal instead of 45° as commonly but erroneously assumed by designers.

The multifarious difficulties with the three bases of inductive reasoning presented in the paper and upon which erroneous code rules and committee recommendations are based, are as follows:

1.—The stress mechanism of the homogeneous and reinforced concrete beams are nearly antipodal, and inductive reasoning from opposites tends to invite error in the conclusions drawn.

2.—Truss analogies are misleading because, while the greatest web stress in the truss occurs next the support at the locus of greatest external shear force, in the beam whether steel or concrete on the contrary the location of the greatest web stress is unrelated to the location of the greatest external shear force, but is fixed by the internal stress mechanism and varies in position from near the support for the short deep beam to near the center of the shallow beam.

3.—General reasoning from the locus of cracks back to elastic resistance involves both the variation of location dependent upon the steel percentage and ratios of depth to span length combined with the practical consideration that the concrete should be designed to remain intact and uncracked under repetition of working loads; hence the laws of elastic resistance should be applied to secure this desideratum.

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* "Reinforced Concrete", by Turneaure and Maurer, p. 204, Equation (5).

H. E. Eckles,* M. Am. Soc. C. E. (by letter).—The author's clear presentation of this important subject will be very valuable to structural engineers interested in economical and careful design. It is an unfortunate fact that, from the standpoint of design, web reinforcement is very often one of the weakest features of reinforced concrete structures. This is frequently due to confidence in the effectiveness of vertical stirrups which the author's analyses show clearly is unwarranted. He has stated that the function of vertical stirrups is to delay failure which has already become a fact, and that they do not take any stress before this stage is reached. These are facts which have been established from the time of the earliest reliable tests of reinforced concrete beams, and it has been equally well known that these drawbacks do not apply to the use of bent-up bars. It is rather inconsistent to design a structure for a condition of failure which, when reached, will necessitate its being replaced, particularly when it may equally well be designed so as not to reach this stage, by the use of bent-up bars.

The author has very significantly stated that bent-up bars may act as web reinforcement before cracks appear in the concrete, and that they act as well to delay failure afterward, in a manner similar to that claimed for vertical stirrups. The statement is also made that bent-up bars deliver their tension as main reinforcement to the compression side of the beam with the result that questions of diagonal tension are eliminated, and that when diagonal bars are used the unit web shear is one-half that in beams with vertical stirrups. These and other considerations make it highly desirable that careful consideration be given to the use of diagonal bars in preference to vertical stirrups. Bent bars may be made thoroughly effective in a much simpler way than that suggested by the author.

The danger of bent-up bars breaking out at the sides of a beam, or causing a beam to split, is unfortunately exaggerated by laboratory tests of beams to destruction. This danger may be avoided by the use of a short, heavy transverse bar placed on the inside of the bends or hooks. This will keep the bent-up bars away from the sides of the beam; will distribute better the bearing of the bent-up bars on the concrete at the bends; and will tie the concrete together transversely. Short hooks or bends at the ends of this transverse bar may well be adopted in important work, and this avoids two of the principal difficulties in design, namely the use of bends of long radii and the splitting of the concrete at the bends. The exterior bars should be kept straight to avoid the danger of failure shown in Fig. 6. Failure, as shown in Fig. 7, will be avoided by the use of short, heavy transverse bars with hooked ends, if the beam has uniform bearing.

A third difficulty to be met in the use of bent-up bars is that of anchorage. The author's suggestion (Fig. 9(*b*)) to bend the bars sidewise out of the vertical plane is unnecessary and highly objectionable from the standpoint of bending, transporting, storing, and placing the bars in the structure. When main reinforcement must stop over supports an initial slipping of the bar may often be reduced by using end hooks of short radii and a short transverse bar at the bends, rather than hooks of long radii and free ends. Sufficient anchorage in other cases can readily be secured without resorting to unusual bends.

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* Chicago, Ill.

The importance of good anchorage for the main reinforcement was clearly shown in the earliest important tests of reinforced concrete beams. It was then stated that a greater resistance of the beam would be secured against initial failure by shear if, instead of using vertical web reinforcement, the amount of main reinforcement used at critical shear points was increased. The author's analyses suggest this in a different way. Design that will delay initial failure is much to be preferred to that merely delaying final failure in a structure that will have to be replaced if the initial stage is reached. Since the concrete takes all the shear when vertical stirrups are used, it is preferable to take the material ordinarily used for vertical stirrups as additional main reinforcement or bent-up bars. This would lower the unit bond stress of the main reinforcement and the neutral axis of the beam at the critical shear point, and thus reduce the tendency for cracks to develop which are caused by initial failure. At the same time, this would give a greater amount of concrete in the section to resist shear, with the result that initial failure would be delayed longer than when vertical stirrups are used in any amount whatever. The author has stated that the total cross-sectional area of vertical stirrups in one end of a beam will generally equal that of the main reinforcement at the points of both maximum positive and negative moment, from which it will be seen that the amount of steel in the vertical stirrups would increase considerably the main reinforcement if used in this way, and would materially raise the useful limit point of the beam.

The author might have gone further in pointing out the comparatively greater value of diagonal web reinforcement over vertical stirrups. Diagonal bars are much more effective in preventing diagonal cracks in the tension side of the beam for both the positive and negative moment areas, since their direction in both cases is nearly at right angles across such cracks. When the sets of diagonal bars are spaced at a less distance apart than jd, they will be very effective for this purpose; vertical stirrups are of no practical value in preventing diagonal cracks, because of their direction. There is a serious defect in the author's analysis of vertical stirrups by the truss analogy since this assumption leads to the anomalous situation of having tension in the concrete in the direction of the vertical stirrup in the compression area. This has been emphasized by the conclusion that hooks are needed at the ends of vertical stirrups. An examination of Fig. 2 and Fig. 3 will help to make clear how this tension must be erroneously assumed. If any considerable tension exists in any part of the length of a vertical stirrup the concrete there must be stressed in tension beyond its point of failure, a condition which would develop cracks throughout the length of all vertical stirrups which are assumed to be functioning. This has a special significance when considering the section of a vertical stirrup which is above and to the left of a major crack, as shown in Fig. 2 (*a*) and Fig. 3 (*a*). In that area the unit tension in the concrete is that in the vertical stirrup divided by n.

All this raises the question of whether the required tension in the stirrup in the compression area must be assumed to exist only if major diagonal cracks occur there. This, in turn, would require the assumption that major cracks exist throughout the length of the beam to the support. This cannot be the case, however, for such cracks exist only at the point where substantial slippage of the main reinforcement has stopped, or where the main reinforcement is greatly overstressed. Tests do not show the existence of numerous cracks such as these before failure has resulted. In the case of continuous beams no substantial slippage can occur where the main reinforcement extends any distance beyond the point of contraflexure, or is properly bent into the compression area of the beam. This renders the use of vertical stirrups entirely futile in such beams. Reducing the main reinforcement over supports merely increases its unit stress and reduces the shearing resistance of the concrete at the most dangerous point, due to increasing the amount of concrete that is broken up by tension in the tension side of the cross-section of the beam. Differences in the moments of inertia of cross-sections at the supports and at the middle of the beam have little effect on the relative moments at these points, since the points of contraflexure are changed but little by these differences. The danger in this region from improper design is well illustrated by Fig. 6.

After all, the concrete must take the shear very largely in all sections of a beam. Tension is developed in diagonal bars to some extent because their direction is at right angles to the diagonal tension cracks in both continuous and simple beams, and, because of this tension in the tension side of the beam in the diagonal bars which is developed by moment, will be of substantial value if properly spaced.

Their effectiveness is apparent by virtue of their being in the direction of rotation about the centroid of compression at a point such as *A*, Fig. 1 (*h*). This makes them effective at the beginning of stress in the beam, in contradistinction to the lack of such action in vertical stirrups before initial failure takes place.

Reliable tests show that concrete has a shearing value of about two-thirds that of its compressive strength, and good design requires only that tension in the concrete be taken care of by suitable reinforcement. When the specifications are such as to insure this, the author's suggestion that higher unit shearing stresses be allowed, may well be adopted. The suggestion to use a higher unit tension stress in the main reinforcement, however, should be examined very carefully. It would reduce the percentage of main reinforcement and hence the area of concrete under compression. In the end it would result in increasing the dead load, in reducing the very effective area for shear on the compression side, and in producing undesirable effects on bond anchorage.

JACOB FELD,* ASSOC. M. AM. SOC. C. E. (by letter).—In analyzing the action of web reinforcement the author has developed his ideas clearly and has checked his theories by comparison with reinforced concrete beams tested to destruction. He might simplify his analysis of the stresses in reinforced concrete by starting with more fundamental ideas than such expressions as "diagonal tension." All strains can be resolved into two types:

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* Cons. Engr., New York, N. Y.

(*a*) Tension or compression, acting along a line, accompanied by strains of the opposite type acting along the other two axes.

(*b*) Shear which is a strain causing no change in volume and is equivalent to a compression along one axis, a tension along the other axis, and no stress along the third axis.

In other words, all strains must be considered along three axes. Referring to reinforced concrete, tension in the embedded steel rods must be accompanied by compression along the axes at right angles to the rod. Shear stress can be resolved into compression and tension along two axes at right angles to each other. This is the source of the mysterious diagonal tension in concrete beams.

The strength of concrete in compression is greater than in tension. However, all concrete must have some strength in tension, otherwise it could not take shear. The strength of concrete in shear is governed by its strength in tension and the assumption of a beam made of concrete capable of taking shear but not tension, is misleading. This double assumption is impossible.

In resisting compression, concrete fails more quickly by its inability to take care of the resulting tension strains (Poisson's effect) than it does in resisting compression strains. The writer has never seen an example of failure of concrete due to pure compression strains. Such a failure would only occur if it were capable of resisting considerable tension.

In resisting shear no cracks develop until the tension component of the shear overcomes the ability of concrete to resist it. It is very difficult to understand why the measured values of the tension in web reinforcement do not appear until cracks develop, because, if the concrete takes tension in resisting shear, certainly the embedded steel along such lines must also take it. However, when concrete sets, the embedded steel is compressed, and it is more than likely that whatever tension is developed in the web reinforcement before cracks develop does not compensate for the initial compression in such steel due to the setting of concrete.

In deep beams, which give a large section of concrete toward the resistance of shear stress, failure in test specimens is not in shear if the tension bars are well anchored. Failure in many specimens is due to splitting, as is well shown by Figs. 6, 7, and 8, because the compression in the concrete has induced tension stresses at right angles to the axis of the compression and no provision has been made to take such tension. The author's suggestion (Fig. 9(*b*)) that rods be bent into opposite flanges would take care of such tension stresses and would permit higher compression stresses in the concrete before cracks appear in the compression face.

HARDY CROSS,* M. AM. SOC. C. E. (by letter).—The author has treated so many aspects of his subject and has described them so fully that the writer has little hope of adding much by this discussion. Those who have been familiar with Professor Mylrea's extended researches in this field will value highly his recommendations.

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* Prof., Structural Eng., Univ. of Illinois, Urbana, Ill.

The author has brought out clearly the importance of the stress trajectories, or traces of the planes of principal stress, in determining the direction at which cracks first appear. Within limits, however, the direction is necessarily a matter of chance. Until these cracks appear there are lines of tension sloping upward toward the end of a simply supported beam and lines of compression sloping downward toward the support. These may be regarded, respectively, as strings with a strut along the top supporting the load in tension and as tied arches supporting the load in compression all within the beam which supports the load in flexure. The arch action and catenary action which designers are constantly discovering in concrete beams, are, therefore, perfectly real; but they are no more real than they are in timber beams or steel beams; they are different aspects of one and the same phenomenon, and not separate things at all.

As cracks occur along the lines of tension the arches carry more and more of the load. It would now seem as if these arches could not fail in shear; but elongation of the main reinforcement and diagonal shortening of the concrete and especially bond slip—all somewhat similar to rib-shortening as usually conceived in arch design—throw secondary bending into these remaining arches and the thrust lines are now inclined to the arches.

The writer thinks that such conceptions as these are rarely satisfactory. They do, however, have some value in bringing attention to the importance, in dealing with a brittle material, of phenomena which are often thought of as secondary. They lead here to a word of caution in dealing with beams with sloping faces, because of these secondary effects. Because concrete is so brittle, it should be fully reinforced.

There are four methods of studying structural design: Analysis, tests, experience with structures in service, and imagination as to possible methods of failure. In reinforced concrete no one of these alone can be called either the proper method or the scientific method of study; all have been at times neglected or over-emphasized; all have proved at times powerfully illuminating and, at other times, deceptive and discouraging.

Perhaps the chief value of the paper is in correlating these rational, semi-rational, and empirical viewpoints and especially in suggesting the importance of secondary modes of failure, such as crushing at bends or splitting from hooks, the importance of which so many designers fail to realize.

P. Wilhelm Werner,* Assoc. M. Am. Soc. C. E. (by letter).—The writer believes that vertical stirrups, although useful from certain practical points of view, should not be relied upon to resist shear. Under normal conditions, pure shear can generally be cared for easily by the concrete alone. If the diagonal tension in reinforced concrete cannot be taken by the concrete, it should be taken by bent-up bars.

The writer would emphasize the importance, under certain conditions, of providing ample reinforcement crossing the lines of support, of both diagonal bars and lower horizontal steel. Actual beams often show nearly vertical cracks at the lines of support. A really dangerous condition may then occur, especially if the beam is also stressed in tension over its entire cross-section. Diagonal bars are capable of taking a component of the direct tension stress in the beam, whereas stirrups are quite ineffective under such loading.

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* Stockholm, Sweden.

Unless engineers can produce a concrete capable of taking considerable tensile stress, the raising of the stresses in the reinforcing steel must involve a greater risk of cracking. Therefore, although the raising of certain allowed unit stresses in reinforced concrete may be warranted for structures in air, it is doubtful whether it would be wise to adopt a general increase of the same stresses for structures in water. Cracks appearing in structures in air are unsightly, but need not necessarily be a matter involving danger. On the other hand, even small cracks occurring in such structures as are subjected to water pressure from one side constitute a source of serious weakness.

In this connection the writer would call attention to the possibility of grading the allowable unit stresses in relation to the character of the structural element in which the stresses occur. Compare, for example, a single beam with a flat slab supported along two opposite edges. Assuming exactly comparable outer forces in both cases, it seems reasonable (with a brittle material like concrete, which often shows a wide variation in strength) to adopt a lower unit working stress in the beam than in the slab, as in the latter case the influence of a batch of inferior quality would be distributed over a comparatively large width. This principle, of course, applies not only to shearing stresses, but to any stress or stress combination induced. Although the writer is aware that it is hardly practicable to burden standard specifications with stipulations of this nature, he thinks the point is worth considering in practical design, as it involves an economic factor of importance.

The writer would emphasize more distinctly the practical feature mentioned by Professor Mylrea, of providing for a bar spacing in proper relation to the maximum size of the coarse aggregate used. This relation has a special bearing on the design of structures in water, as it may affect the durability of the concrete. A grid of heavy reinforcement must inevitably act somewhat as a screen, retaining the larger stones. Thus, although the concrete may have the proper grading when arriving in the forms, a change in the grading will most likely occur during the pouring, and that on both sides of the reinforcing bars. Consequently, the concrete around the bars may be more or less porous and pervious, and subjected to the deteriorating and corrosive action of percolating water.* Aside from this, however, there seems to be no doubt that crushed stone up to 3 in. in size can be used, even for heavily reinforced concrete, without causing "pockets".

Professor Mylrea's suggestion to use unusual bends for anchorage, etc., although for the present rather objectionable from a practical point of view, may prove sooner or later to be an economical success. Every country is developing a nucleus of highly skilled workmen for reinforced concrete. What is considered as complicated to-day, may prove to be a comparatively simple matter in the future.

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* "Corrosion of Concrete," by John R. Baylis, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 791.

T. D. Mylrea,* M. Am. Soc. C. E. (by letter).—The writer has been impressed with the view, expressed by several contributors, that cracking and failure of reinforced concrete beams are synonymous. This is not necessarily the case. The occurrence of fine cracks means simply that the reinforcement is being stressed, and if reinforcement is present in the proper amount and distribution, such cracks need cause no apprehension. Prominent cracks may indicate one or more of several things. Wide vertical cracks, or more often, one wide crack in a region of low shear usually accompany stress beyond the yield point in the reinforcement. In simple beams they may indicate slip of the main reinforcement. Wide diagonal cracks between the supports and mid-points of simple beams generally mean slip of bar or crushing of concrete under the bends of the web reinforcement. These cracks are particularly dangerous, for they show that the load limit of the beam is reached, and that real failure, even under a decreasing load, may follow quickly. Thus, cracking may or may not mean danger. The cracking is not necessarily dangerous, if the reinforcement is adequate.

Mr. Godfrey has presented a number of interesting points. For example, he questions the validity of the 45° assumption in the development of the formulas for web reinforcement. If a reinforced concrete beam were truly homogeneous, such cracks as occurred would tend to form at right angles to the tension stress trajectories. If the beam could be modified so that the horizontal components along all tensile stress trajectories were resisted by the horizontal steel, then the diagonal tension and diagonal compression components of the vertical shear on any vertical section would be a maximum on planes inclined at 45° to the vertical, and cracks above the steel and below the neutral axis would occur at an angle of 45 degrees. In regions of low vertical shear, where they are not dangerous, cracks would be slightly inclined. It is a fact that in simple beams cracks generally occur in this manner. Where, owing to slip of the main reinforcement, a dangerous diagonal crack occurs, the crack is more nearly horizontal for some distance along the bar, gradually approaching 45°, and then tending toward the horizontal again. Even here, where vertical stirrups would be of little help, the 45° assumption as to the direction of crack is on the safe side.

In continuous beams the stress trajectories become horizontal over the supports, and vertical cracks may form in the top of the beam at the face of the support. Within the angle formed by the face of the support and a line extending at 45° from the junction of the lower surface of the beam and the face of support, cracks will tend to form at angles between the vertical and 45 degrees. Fine cracks within the 45° line are not necessarily dangerous, for, as Mr. Godfrey states, "with the top and bottom rods firmly anchored into the support, a large shear can be carried by the concrete". The resistance to the formation and development of these cracks is furnished by the horizontal negative reinforcement, not by the web steel. If the negative reinforcement is deficient in amount or in anchorage into the support or out into the beam proper, these cracks will develop, and no type of web reinforcement will prevent trouble. This, however, is no concern of the web reinforcement, for it can only reinforce the web outside the 45° line. Thus, again, the 45° assumption seems not erroneous for use in the design of the web steel. Mr. Godfrey mentions the weakness at this point, and so does Mr. Werner, but if the horizontal steel were deficient the diagonal bars recommended by Mr. Godfrey could function only by sidewise bearing in the concrete, which he, himself, points out as being ineffective. It seems to the writer, then, that while Mr. Godfrey's method of web reinforcement is undoubtedly a good one, the 45° assumption is satisfactory as a basis for working formulas.

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* Prof., Bldg. Constr., Carnegie Inst. of Tech., Pittsburgh, Pa.

Mr. Godfrey also mentions the difficulty of anchoring bars properly and compares an anchorage to the end detail of a truss member. In this the writer heartily concurs; but because he had presented a former paper* dealing exclusively with this subject, he did not develop the point further at this time.

The writer has observed a curious phenomenon in simple beams tested to destruction. When uniform loads were used, the dangerous diagonal tension crack formed at about the quarter-point of the span, whereas when concentrated loads were used the important crack ran diagonally upward from the reinforcement toward the load point at an angle of about 45 degrees. Mr. Turner's discussion may hold the clue to this phenomenon. It seems to the writer that if stirrups are necessary at all in simple beams they should be carried at least to the quarter-points.

It was not the writer's intention to bring bi-axial strain into the discussion, as suggested by Mr. Feld, for he feels that web reinforcement can be designed effectively and economically without such complication. Professor Cross has well summarized some of the writer's ideas in the last paragraph of his discussion; but it is the writer's intention also to point out that when steel of a high elastic limit is used, great progress is still possible in reinforced concrete design. If the designer wishes to take advantage of these possibilities, he must constantly keep in mind all the precautions pointed out, even if, as Mr. Eckles states, some of them may appear to be extreme.

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* *Journal,* Western Soc. of Engrs., January, 1926.

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Paper No. 1744

EXPERIMENTS ON DISCHARGE OVER SPILLWAYS AND MODELS, KEOKUK DAM*

By Floyd A. Nagler† and Albion Davis,‡ Members, Am. Soc. C. E.

With Discussion by Messrs. B. F. Jakobsen, J. B. Spiegel, Dana M. Wood, S. P. Wing, E. A. Rudulph, C. H. Pierce, Ludwig A. Ott, B. F. Groat, and Floyd A. Nagler and Albion Davis.

Synopsis and Conclusions

This paper describes rather unusual discharge measurements made on the crest of the spillways of the Keokuk Dam, in Iowa. These studies were supplemented by tests made on a laboratory model, in order to determine a more accurate spillway rating than had formerly been available and in order to make possible a final and precise revision of the daily flow records of the Mississippi River at this point. Volumes of flow averaging about 4 000 sec-ft. were measured in the 30 ft. of spillway width, at heads slightly less than 11 ft.

Unless one has attempted to measure water running 24 ft. per sec. he cannot appreciate all the difficulties attendant on an accurate spillway crest gauging under a head of 11 ft. Special equipment was required to handle properly the stay-wires supporting the meters. By this means it was possible to make about 150 velocity observations in each gauging, fully exploring the entire cross-section up to within 0.4 ft. of the spillway edge. It was also possible to measure all the vertical and horizontal angles of flow and to rate the meters so as to obtain results that were accurate within 2 per cent.

It was found that opening the gates adjacent to the spillway where the water was being measured had a very considerable effect on the discharge coefficients. In the formula, $Q = C L H^{\frac{3}{2}}$, the coefficient, C, for the normal operating head* of 10.8 ft., varied from 3.71, when a single spillway was discharging, up to 3.90, when many spillways were discharging. The gaugings also showed clearly that, with a crest unobstructed by piers, C, for a 10.8-ft. head, increased to 4.00.

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* Published in February, 1929, *Proceedings*.

† Prof., Hydr. Eng., Univ. of Iowa, Iowa City, Iowa.

‡ Chf. Engr., Mississippi River Power Co., Keokuk, Iowa.

The coefficients developed by these measurements are in rather striking contrast to the conservative coefficient of 3.33 that was used in providing spillway capacity in the Keokuk Dam, notwithstanding the fact that in designing the dam it was realized that the actual coefficients would probably be considerably greater than 3.33. The use of the new spillway rating yields flood flows that are in some cases 10% in excess of the former determination.

The field measurements were followed by a study of the behavior of a model of three of the spillways built to a scale of one-eleventh the size of the Keokuk Dam in the Hydraulic Laboratory of the State University of Iowa.

These experiments duplicated the results observed at the dam itself under normal conditions with remarkable reliability, and studies were made to determine the probable behavior of the Keokuk spillways when discharging under abnormal pond levels.

Experiments were performed on the model when its surface was both smooth and rough, and with and without a fill of sediment up stream from the dam. It was demonstrated that, in experimenting with such a large model, the relative amount of friction caused by the varying surface conditions of the model was of small significance.

Several pier shapes differing from those used at Keokuk were constructed on the model. Extending the pier nose up stream, or giving it a tapered form so that it attained its full thickness a short distance down stream from the highest part of the spillway crest, produced a dam with a greater discharge than that obtained with the Keokuk design.

HISTORY OF THE CALIBRATION OF THE SPILLWAYS

Both prior and subsequent to the actual construction of the water power development, the engineers of the Mississippi River Power Company have kept continuous records of daily flow at Keokuk, as well as at other points on the Upper Mississippi River.

The flow record at Keokuk subsequent to the construction of the dam has been computed with considerable accuracy by calculating the discharge through the water turbines, and the quantity of surplus water discharged over the spillways. Field measurements of the hydraulic efficiency of the various turbine units were available. These served as a basis for accurate computations of turbine discharge.

Pond elevations taken every hour and a record of gate operations permit computation of the flow over the spillways by the use of a weir formula. The coefficients for this formula to compensate properly for the nappe contractions over the crest and around the piers, however, must be determined definitely by measurements if an accurate record of discharge over the dam is to be obtained.

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* Throughout this paper, H in this formula denotes the depth of the pond water above the highest part of the concrete crest.

Previous to 1924, coefficients had been used which were based partly on some early measurements made on the discharge through single spillways at times when the pond level was maintained at an elevation of less than 5.28 ft. above the crest, and also on a rating curve for these spillways prepared by John R. Freeman, Past-President, Am. Soc. C. E., and derived from experiments on models of the Croton Dam. The normal operating head on the Keokuk spillway is 11 ft., a head much in excess of that existing in any of these tests, and much higher than had existed in any measurements known to the writers.

For some time it was feared that the coefficients obtained in this manner were too small for use under heads as great as 11 ft., perhaps to the extent of 10%, but no change was made from the original rating so that all records of daily flow would be comparable. From the daily computations of flow over a period of several years, and from current-meter measurements of flood discharge above and below the dam, several fairly definite conclusions were developed. It was noted that there was a tendency for the computed flow at Keokuk to be lower on Sundays than during week days when there had been no apparent change in natural flow entering the pond. The only difference in Sunday operation was that more water passed over the spillways and less through the water turbines than on contiguous week days. Flood discharges computed from the several measurements in the open river invariably indicated a higher flow than those computed from the records of plant and spillway operation. It became evident that a new and more nearly correct rating of the spillways must be obtained if the Keokuk records were to represent the true discharge at that point. The tests described herein were undertaken as the first step toward making the flow records available for publication for the benefit of the profession.

The co-operation of the engineers of the Hydraulic Laboratory at the State University of Iowa was enlisted in obtaining the actual measurements of flow through the Keokuk spillways. The Laboratory Staff was interested in undertaking this work as a research problem, not only because of the value to the Engineering Profession of the actual field measurements of discharge over the Keokuk spillway, but also because of the opportunity afforded thereby to make comparisons between the large-scale results and the performance of models.

Eight measurements of the discharge over the spillways were made by operating Ott current meters in the cross-section of the nappe above the crest during the period from April 29 to May 8, 1924.

A model of three sections of the spillway one-eleventh of the full size was constructed in the testing canal of the University of Iowa Hydraulic Laboratory, and studies were made during the period from September 16 to October 29, 1925.

Before removing the model from the testing flume, an investigation was made to compare the contraction effects of certain other forms of piers with those of the Keokuk design. These experiments were performed during the period from November 19, 1925, to May 5, 1926.

Description of Spillway

The actual dam consists of 119 spillway sections extending from the Illinois shore to the up-stream corner of the power house. A view of the spillway is shown in Fig. 1. Each spillway section is 30 ft. wide with a crest about 30 ft. above the former natural stream bed. The spillways are separated by piers which are 6 ft. thick, with a cross-section at the up-stream end in the form of a semi-circle (see Fig. 2).

The typical cross-section of the spillway shown in Fig. 3 gives the details and dimensions of the form of the crest. Soundings made up stream from the spillways selected for the tests indicated that the bed of the stream approaching the dam is much higher than the natural stream bed at these points. This is due in some cases to coffer-dam material left in place and, in other cases, to natural sedimentation. In the twelve years since the construction of the dam, there has been considerable silting in front of those gates that are operated very infrequently and where "dead water" exists nearly all the time. In front of the gates that are operated more frequently, however, there is practically no sedimentation.

Fig. 2, which is a view of the up-stream face of the dam, shows the well-rounded approach to the crest of the spillway. Extending from the concrete crest upward there is a slot 21 in. wide and 14 in. deep in the face of each pier, in which steel gates, 11 ft. high and 32 ft. wide, are operated. The water level behind the dam is maintained at all times close to the top of the gate, which is about 11 ft. above the crest of the dam. The steel gates, which are of the plain sliding variety without rollers, are finished on the bottom with a timber sill which makes the water seal against the carefully finished concrete crest of the dam. The gates are raised by electrically operated traveling cranes, and are always lifted clear of the water surface, so that a spillway discharges to its full capacity. What little leakage there is when a gate is lowered, is cut off effectively by a few shovelfuls of cinders, when the leakage becomes a factor in the operation of the plant.

The spillways are numbered in order from the Illinois shore toward the power house. The discharge measurements were made on Spillways Nos. 68 and 103. Spillway No. 68 was chosen as representative of those near the center of the dam, and Spillway No. 103 was selected from among those nearer the power house where the pond is deeper.

Current Meter Discharge Measurements at Keokuk Dam, April 29 to May 8, 1924

After a careful study had been made of the various possible methods of accurately measuring the discharge through the spillway, it was decided to operate current meters in the nappe as it flowed over the crest of the dam. It was appreciated, however, that stream velocities would be nearly 25 ft. per sec., which is greatly in excess of speeds ordinarily encountered in current meter practice; and that, in parts of the nappe, stream filaments would approach a rigidly supported meter at a considerable angle with the meter axis. Non-rigid suspension of metering equipment was entirely out of the question.

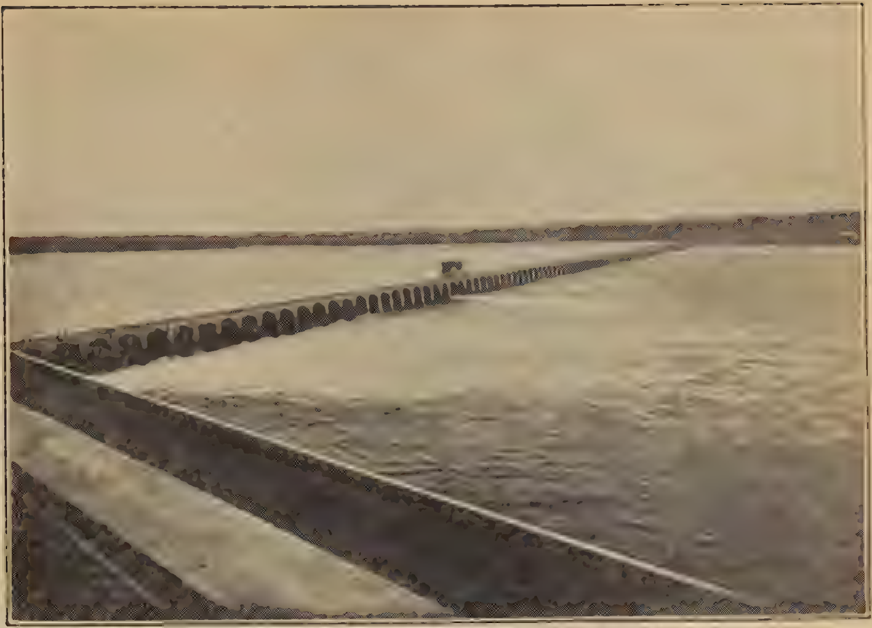


FIG. 1.—VIEW OF KEOKUK SPILLWAY FROM POWER HOUSE.

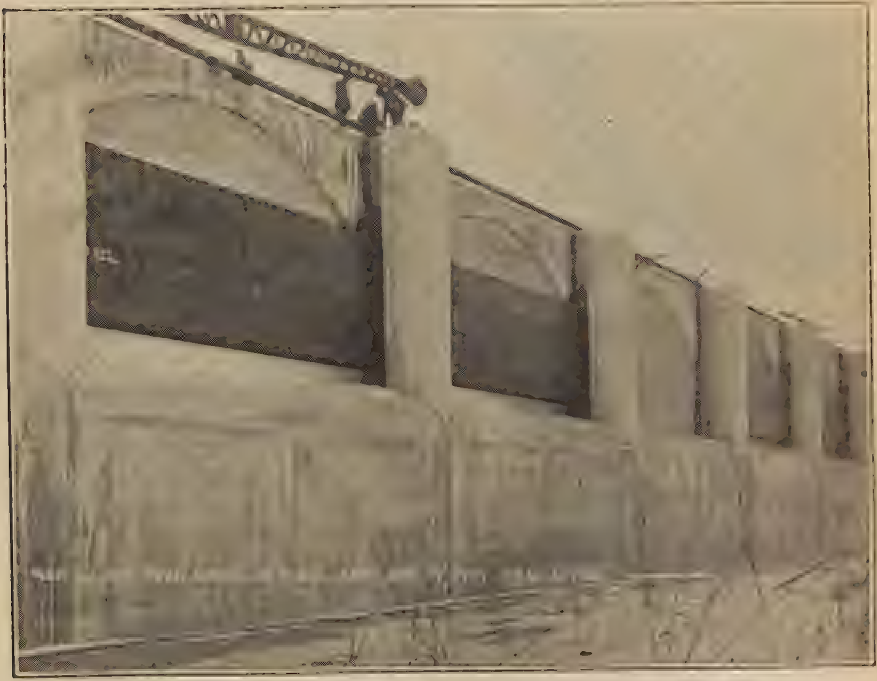
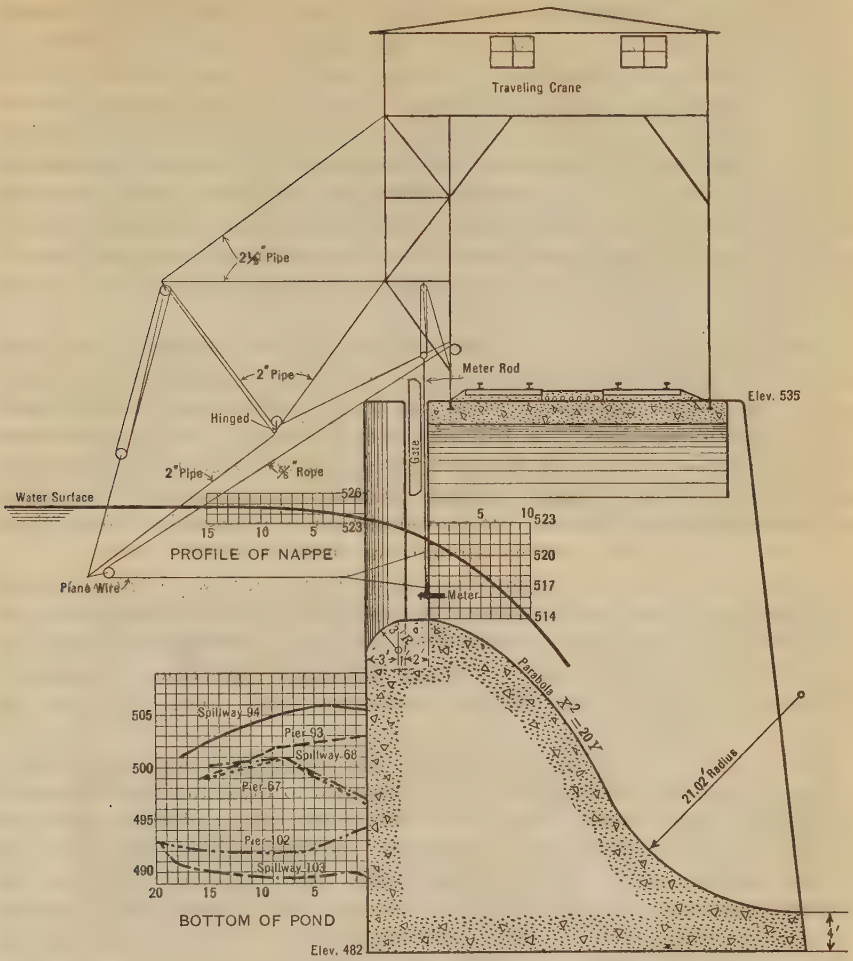
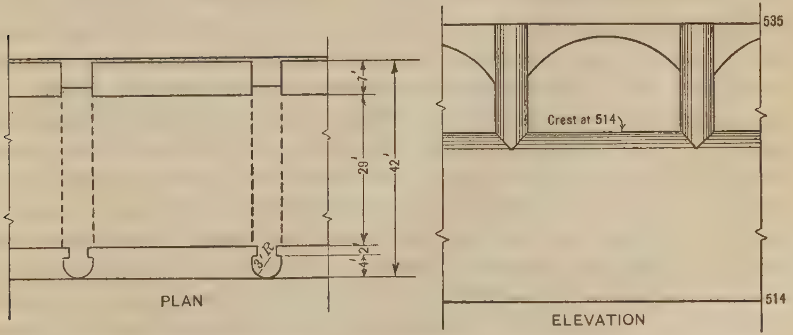


FIG. 2.—UP-STREAM FACE OF KEOKUK SPILLWAY BEFORE POND WAS FILLED, FEBRUARY 6, 1913.


Traveling Crane
2½" Pipe
2" Pipe
Hinged
2" Pipe
⅝" Rope
Meter Rod
Gate
Elev. 535
Water Surface
526
523
15
10
5
PROFILE OF NAPPE
Piano Wire
5
10
523
520
517
514
Meter
3' R
3
1
2
Parabola X² = 20Y
21.02' Radius
505
500
495
490
Spillway 94
Pier 93
Spillway 68
Pier 67
Pier 102
Spillway 103
20
15
10
5
BOTTOM OF POND
Elev. 482
4'

SECTION SHOWING EQUIPMENT FOR HANDLING CURRENT METER


7'
29'
42'
2'
4'
3' R
535
Crest at 514
514

PLAN

ELEVATION

FIG. 3.—DETAILS OF SPILLWAYS IN THE MAIN DAM.

Development of Method of Measurement.—The Ott type of current meter seemed admirably adapted for these measurements, not only because it measures the normal component of angular currents more accurately than other types, but also because the bearing at the end of its horizontal axis is capable of taking the strong horizontal thrust caused by these high velocities. A Price meter was used successfully in one attempt to measure the velocities all the way through the nappe to the crest, but considerable wear was noted on the pivot point and sides of the cone bearing. This indicated that the pivot shaft was rubbing on the metal side of the bearing and that this type of vertical shaft meter could not long withstand the thrust from high velocities of current.

The section of the nappe between the gate slots was selected as the more economical and the safest place for measurement, but to hold a meter steadily against the forces encountered when the water flowed under 11 ft. of head proved to be no small task. A 1-in. pipe with a fulcrum 4 ft. above the water could be immersed only 3 ft. before it was permanently bent. When a 2-in. pipe was tried with a similar fulcrum, it could be lowered but 6 ft. before permanent deflection occurred. It proved absolutely impossible to force these rods to the crest of the spillway through the sheet of moving water.

After considerable experimentation and deliberation, it was decided to hold the lower end of the rod in position by means of a stay-wire attached to an adjustable and movable framework extending up stream from the dam. It was considered too dangerous to attempt the meter work from a temporary platform built close to the swiftly moving water at any lower section of the nappe.

Metering Equipment.—A framework made of standard pipe was attached to one of the two traveling cranes which are used to raise and lower the spillway gates. Figs. 3 and 4 show the details of the design of this framework and its relation to the metering operations. By means of the traveling crane it was possible to move the framework to any lateral position relative to the spillway, and the outer end of the lower section of the framework to which the stay-line pulley was attached could be lowered or raised to any desired depth. The stay-wire that passed over the pulley and through the water was divided about 5 ft. in front of the measuring section in order to support the rod in two places. It was made of No. 14 piano wire and could be lengthened or shortened as desired. By proper adjustment of the position of the traveler, location of the swinging pipe frame, and the length of the stay-wire, the meter rod could be kept in perfect vertical alignment in the metering section.

The meters were attached to the bottom of steel rods, 23.5 ft. long, which were either 1 or 1.25 in. in diameter, depending on which meter was in use. The stay-wires were connected to the rod by collars inside of which the rod was free to rotate. However, the rods were provided with keys to keep the meters and rod handles from rotating about the shaft. The meters were raised and lowered with the rod by hand or with the aid of a pulley and rope attached to the frame of the crane. The meters and rods could be lowered into position by passing them through the 5-in. space between the downstream side of the gate and the up-stream face of the concrete deck of the

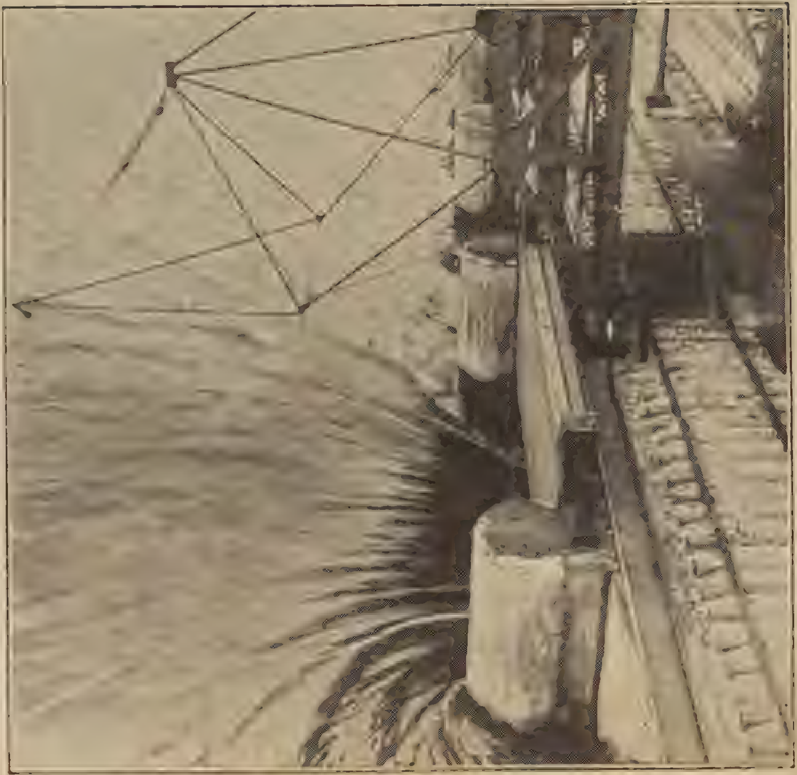


FIG. 4.—FRAMEWORK FOR HANGING STAY-WIRES FOR THE SUPPORT OF THE CURRENT METER ROD.

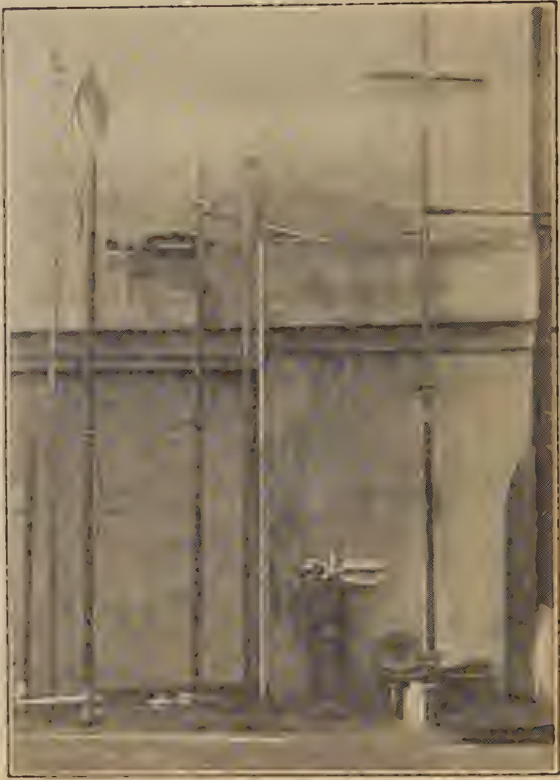


FIG. 5.—METERING EQUIPMENT USED IN KYOTO MEASUREMENTS.

dam. Guides at the top and bottom of the gate were used to plumb the rod at designated verticals in the measuring section.

Current Meters.—Three meters of the Ott type were used in making the different measurements, although the equipment permitted the operation of but one meter at a time. (See Fig. 5.) Most of the work was done with a small Ott meter (No. 2908) which had a propeller with two blades. Soon after the work started, this meter was equipped with a special tail-piece and a guard to protect it against striking the face of the concrete pier. With this instrument it was possible to make velocity measurements within 0.4 ft. of the side of the spillway. A steel rod, 1 in. in diameter, was used and this whole equipment seemed easier to handle and keep under control than that used with the larger meters.

The Mensing-Ott meter (No. 3246) was used with a 3-blade propeller, and also with a 2-blade propeller. With the 3-blade propeller it had characteristics in measuring angular currents which were superior to the others, but because of the wheel it was lowered with difficulty in the small space between the back of the gate and the concrete deck of the dam, and because of the larger rod, was not as easily handled in the swiftly moving water as the smaller meter. It did not prove practical to make measurements closer than 2 ft. from the pier with the larger meter because of the danger of the rod becoming unmanageable in moving from one section to another in the oblique current and striking the pier. At one time, when the rod got out of control, the meter struck the pier, but the propeller escaped damage because of the recess at the gate slot. When the larger meter was used, the part of the discharge nearer to the piers than 2 ft. was measured with the small meter provided with a steel guard.

While the vertical rod held the meter in a horizontal plane at all times, no attempt was made to keep the meter axis in a line perpendicular to the measuring section. The meters were supplied with longer tail-pieces than are ordinarily used, and were allowed to swing into the thread of the current. Throughout the major part of the spillway the vertical plane through the thread of the current was perpendicular to the metering section, but near the piers there was considerable contraction of the nappe, with resultant oblique flow. The horizontal angle assumed by the meter axis was measured for every point in order to correct the velocity measurements for this deflection. A handle clamped firmly to the rod near its upper end furnished the necessary index for measuring this deflection.

Much experimentation was done with various forms of tail-pieces and methods of attaching the stay-wire to the meter rod. On four different occasions the piano wires broke despite the care taken in their preparation. Once, when the lake was rather rough, the twist on the meter rod became severe enough to shear off the screws holding the key in place. During the first three days of the tests the bending of the rods was a daily occurrence. In every case of trouble, however, the man operating the rod was quick enough to prevent losing or seriously damaging any of the equipment.

The strain of operating the small meter in the high velocities was so great that during the tests it proved necessary to make a new steel shaft for this instrument due to the excessive wear that had taken place in the ball races of the guide-bearing near the propeller.

A special device was constructed to measure the vertical as well as the horizontal angle of the current in the nappe. It was operated from the up-stream side of the gate and was constructed so that the center of the tail-piece stood in the plane of the measuring section. The tail-piece was about 2 ft. long and was hung about 5 in. down stream from the 1-in. pipe which supported it. The up-stream end of the tail-piece was pinned to a $\frac{3}{8}$-in. iron rod which passed inside the 1-in. pipe and extended through a hole in a tee at the top. All parts adjacent to the vane were thinned down and given stream-line surfaces to offer the least resistance to flow. Handles for operating the equipment were screwed to the tee. The pivot point was located so that the weight of the tail-piece balanced the weight of the $\frac{3}{8}$-in. iron rod. The stay-wires were connected to the pipe in a manner which left it free to rotate. Horizontal angles were indicated by the angle which the handles made with the face of the gate; the vertical displacement of the $\frac{3}{8}$-in. rod through the tee at the top gave the data required to compute the sine of the vertical angle of the current. The equipment was very rugged, but, nevertheless, dependable and accurate.

Rating of Current Meters.—During the tests the water attained velocities of 13 to 26 ft. per sec. and struck the meter at angles with the axis of 0 to 22 degrees. It was impossible to secure ratings in velocities as high as most of those encountered, but after a survey of the possibilities at various rating stations, the meters were sent to the U. S. Bureau of Standards where they were calibrated in velocities as high as 16 ft. per sec. Since the bearing friction becomes a negligible force at high velocities of flow, the rating curves for all these meters were straight lines at velocities greater than 2.5 ft. per sec. They were thus extended to give the rating throughout the range of velocities encountered. The meters were rated both before and immediately following the measurements at Keokuk. A special rating of Meter No. 2908 was made with the guard mounted on the meter frame. While no part of this guard was within 2 in. of the propeller, this obstruction behind the wheel retarded the revolution approximately 2 per cent.

The meters were also rated with the water approaching the propeller at horizontal and vertical angles. These ratings showed little if any difference in the registration, with currents approaching at the same angles from various directions. In streams of great obliquity the three Ott meters under-registered the axial component of the currents by different amounts, the 3-blade meter having by far the best characteristics. Fig. 6 gives the amount of the under-registration in currents of high velocity as obtained from the ratings at various angles. In other words, it shows the correction (in percentage) to be added to the measured velocity in order to obtain the actual component of velocity along the axis of the meter. The meters used were rated by the U. S. Bureau of Standards during April and June, 1924.

Discharge Measurements.—Three discharge measurements (Tests 1, 2, and 8, in Table 1), were made when the spillway being metered was remotely isolated from others that were open. Column (4) of Table 1 gives the location of the tests. Test 1 was performed merely to familiarize the observers with the operation of the equipment.

Tests 3 to 7, inclusive, were performed on Spillway No. 68 in order to determine the effect of the operation of adjacent spillways on the discharge through the spillway under test. Thus, in Test 3, the adjacent spillway, No. 69, was open. In Tests 4 and 5, one spillway on each side of No. 68 was opened. These two tests were performed under exactly the same spillway conditions, but with different meters, in order to check the accuracy of this work; also, because parts of Test 4 were performed on different days, it was thought desirable to repeat this test in a continuous run. In Test 6, the spillway observed was the end one of a series of five that were open, and in Test 7 the measurements were made on No. 68 when operating as the center spillway of a series of seven which were discharging fully.

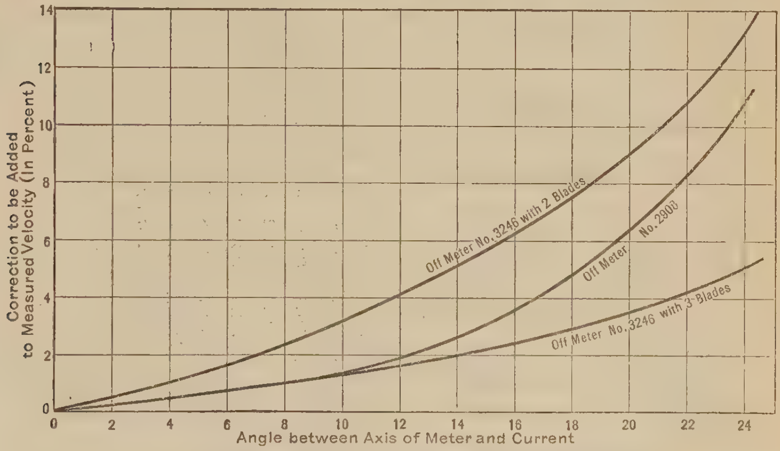


FIG. 6.—CORRECTION TO BE ADDED TO MEASURED VELOCITY.

Individual velocity measurements adequate in number and location to determine definitely the vertical variation in speed of current were taken in vertical lines located at the following distances, in feet, from the side of the spillway: 0.4, 1.0, 2.0, 3.5, 5.0, 7.0, 11.0, 15.0, 19.0, 23.0, 25.0, 26.5, 28.0, 29.0, and 29.6.

The meter revolutions ordinarily were measured at points 1 ft. apart in each vertical, except near the spillway crest, the water surface, and in the shallower verticals near the edge of the spillway, where observations were made at ½-ft. intervals. The period during which the meter revolutions were counted was generally between 30 and 60 sec.

The section in which the metering was done proved to be an ideal one in many respects. The water being accelerated as it passed over the crest, flowed with remarkable uniformity in smooth stream lines and was free from the characteristic pulsations in velocity and annoying vortices which so often accompany the phenomenon of flow in other channels. Many checks of observations were made at certain points after short lapses of time, with the invariable result of duplicating the previous readings exactly or within slight

TABLE 1.—GENERAL SUMMARY OF RESULTS OF KEOKUK SPILLWAY CALIBRATION, APRIL 29 TO MAY 8, 1924.

Test No. (1)	Date of test. (2)	Total duration of test, in hours and minutes. (3)	No. of gate tested. (4)	Average elevation of spillway crest, in feet. (5)	Nos. of adjacent gates open. (6)	Meter used on Stations 2 to 28. (7)	Meter used on Stations 0.4, 1, 29, and 29.6. (8)	AVERAGE POND ELEVATION, IN FEET. East gauge. (9)	West gauge. (10)	Weighted average. (11)	Maximum pond elevation, in feet. (12)	Minimum pond elevation, in feet. (13)	Variation in pond elevation, in feet. (14)	Area of gauging section, in square feet. (15)	Average velocity in gauging section, in feet per second. (16)	Measured discharge, in cubic feet per second. (17)	Head on crest of spillway, in feet. (18)	Coefficient of discharge, C, in $Q = C L H^{\frac{3}{2}}$. (19)	Discharge for Pond Elevation 524.80, in cubic feet per second. (20)
1	April 29..	7–52	68	513.97	None	Small 2-blade	From Test 8	524.86	524.87	524.86	524.91	524.82	0.09	229.0	17.23	3 946	10.89	3.66	3 913
2	April 30.. May 1..	5–30	68	513.97	None	Large 2-blade	From Test 8	524.82	524.83	524.83	524.91	524.73	0.18	230.1	17.47	4 020	10.86	3.74	4 004
3	May 2.. May 3..	9–16	68	513.97	69	Large 2-blade	Small 2-blade	524.82	524.81	524.81	524.91	524.73	0.18	232.7	17.21	4 006	10.84	3.74	4 001
4	May 3.. May 5..	4–18	68	513.97	67–69	Small 2-blade guarded		524.73	524.75	524.72	524.86	524.55	0.31	231.8	17.31	4 002	10.75	3.78	4 046
5	May 5..	4–31	68	513.97	67–69	Large 3-blade	From Test 4	524.56	524.58	524.59	524.74	524.39	0.35	234.6	16.56	3 918	10.62	3.78	4 041
6	May 6..	4–18	68	513.97	69–70 71–72	Small 2-blade guarded		524.76	524.77	524.78	524.83	524.66	0.17	226.0	17.54	3 964	10.81	3.72	3 974
7	May 7..	4–40	68	513.97	65–66–67 69–70–71	Small 2-blade guarded		524.95	524.97	524.96	525.01	524.92	0.09	237.7	17.94	4 264	10.99	3.90	4 172
8	May 8..	3–39	103	513.89	None	Small 2-blade guarded		524.87	524.88	524.87	524.94	524.82	0.12	231.0	17.51	4 044	10.98	3.71	4 006

differences of not more than a fraction of a second. The variation in velocity from the water surface to the crest also was remarkably uniform. In many verticals, a long section of the velocity curve was almost a straight line. This insured the determination of this curve to a high degree of accuracy.

Nappe Cross-Section.—The physical dimensions of the spillways were measured at the dam. The upper surface of the nappe at the section where the meter propeller was operating was determined at each vertical by taking a rod reading with the propeller at the water surface. As a check on these readings the surface was also measured with a steel tape, at sections under both the up-stream and the down-stream side of the gate before and after each discharge measurement. The three sections thus obtained during Test 8 are shown in Fig. 7, the metering section being about midway between the other two. At times of little wind there was no difficulty in determining the water surface of the nappe at the point of measurement within a few hundredths of a foot, and under no condition were there surface waves of sufficient magnitude to render this measurement uncertain by as much as 0.5% of the total depth.

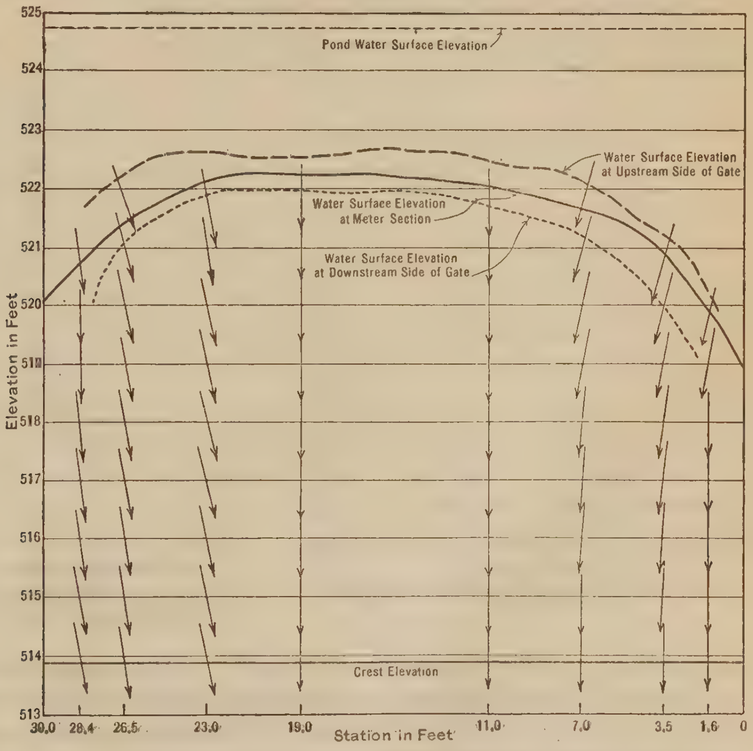


FIG. 7.—HORIZONTAL ANGLE OF CURRENTS OBTAINED DURING TEST 8.

Pond-Water Level Measurements.—As 3 to 9 hours were required to make a single measurement of discharge it was fortunate that the lake was large and little variation in pond level occurred during the tests. The maximum difference in pond level of 0.35 ft. occurred during Test 5, but the average maximum for the eight tests was only one-half this difference. (See Table 1.)

Elevations of the pond-water surface were recorded every 15 min. during the test by Gurley printing water-stage registers located on Piers No. 30 and No. 90 and close to Pier No. 119. In addition, the water level at points near certain closed spillways between the two recording gauges were read with a tape and weight in order to detect the amount of depression in the pond-water surface approaching the open spillways under test.

Obliquity of Currents in Nappe.—Using the current direction meter, measurements of the vertical as well as the horizontal angle of the current were made in connection with Test 8 on Spillway No. 103 discharging alone. These readings were then repeated on this same spillway (as Test 9) with the nappe greatly contracted on one side, caused by the opening of Spillways Nos. 104 to 109 during the test. Readings were taken at most of the verticals used in the current-meter work and at intervals of 1 ft. in elevation.

In Test 8, the maximum horizontal angle of approach of the current (18° 30′) was observed at the surface 3.5 ft. from the pier. The horizontal angles observed at other points are shown graphically in Fig. 7.

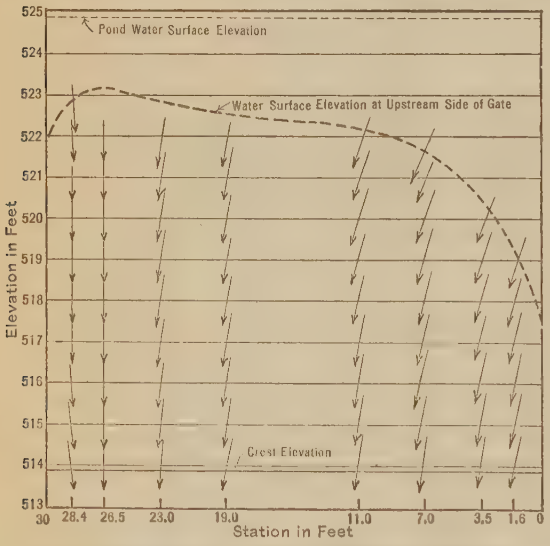


FIG. 8.—HORIZONTAL ANGLE OF CURRENTS OBTAINED DURING TEST 9.

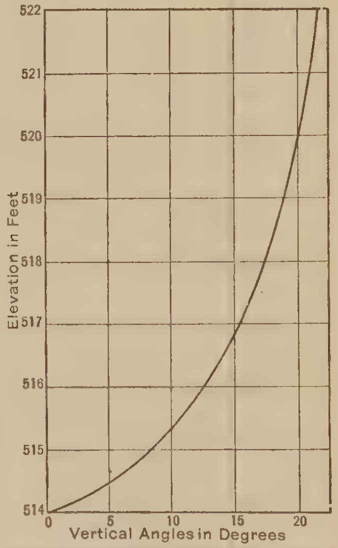


FIG. 9.—AVERAGE VERTICAL ANGLE OF CURRENTS AT VARIOUS ELEVATIONS, TESTS 8 AND 9.

A large contraction of the nappe around Pier No. 102 existed in Test 9, giving a maximum horizontal approach of the current equal to 22° 10′ at the water surface 7 ft. from this pier. The horizontal angles observed at other points are shown graphically in Fig. 8.

The vertical angles at which the current approached the metering section were remarkably constant at a given elevation above the spillway crest. The average results from the observations in Tests 8 and 9 are shown in Fig. 9. Only twelve of the readings taken in Tests 8 and 9 gave values which were more than 3° from this curve, and in no case was there a departure of more than 5 degrees. This average curve was used as the basis for all computations involving the evaluation of the vertical angle of approach of the water toward the meter.

Computation of Actual and Normal Current Velocities.—Meters, held rigidly on a vertical rod that was free to swing with the current, registered only the component of the stream velocity in the horizontal plane passing through the meter. If the vertical angle of approach was significant, even the value of this component was under-registered by the Ott meter. The treatment of the measured current values in one of the fifteen verticals constituting Test 8, is illustrated by Fig. 10 and the data are shown in Table 2.

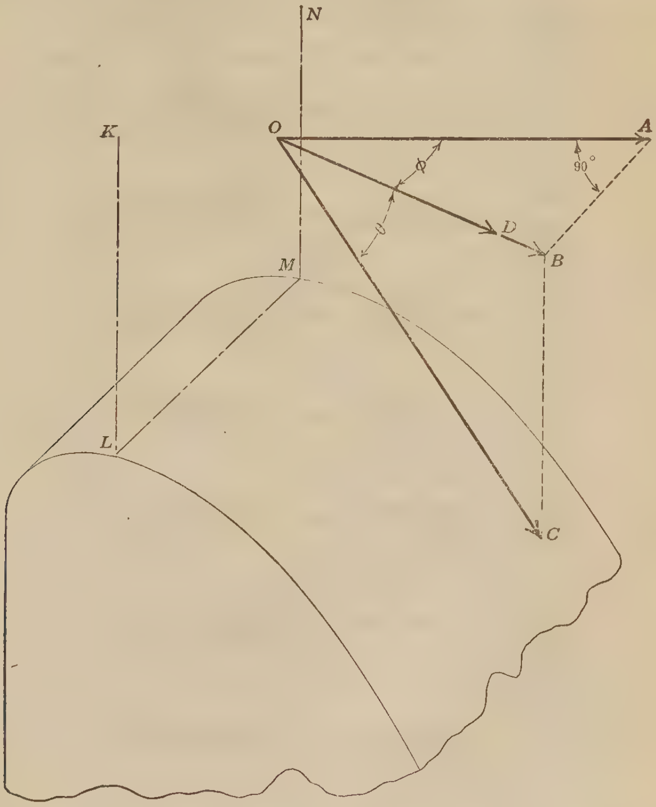


FIG. 10.

These ratings were made with Meter No. 2908 at Station 23.0. It was rigidly clamped to a vertical rod free to swing about its vertical axis so that the meter axis lay within a vertical plane passing through the thread of the current with the propeller located at Point O in Fig. 10. In measuring the velocity of the current passing in the direction, $O\,C$, the meter should have registered its component, $O\,B$, projected vertically into the horizontal plane passing through the axis of the meter. The ratings showed that as a matter

TABLE 2.—CURRENT VELOCITIES, KEOKUK SPILLWAY RATINGS.

Elevation of meter, in feet. (1)	Vertical angle. (2)	METER. Revolutions. (3)	METER. Seconds. (4)	Measured velocity, in feet per second. (5)	Meter corrected for angle. (6)	Horizontal velocity, in feet per second. (7)	Secant, vertical angle. (8)	Actual velocity, in feet per second. (9)	Measured sine, horizontal angle. (10)	Horizontal angle. (11)	Cosine, horizontal angle. (12)	Normal velocity, in feet per second. (13)
514.28	3°3′	1 000	38.9	22.47	1.002	22.51	1.002	22.56	0.24	13°50′	0.971	21.86
514.67	6 20	1 000	40.2	21.43	1.006	21.56	1.006	21.69	0.20	11 30	0.980	21.13
515.17	9 00	750	31.9	20.54	1.011	20.77	1.013	21.04	0.15	8 40	0.989	20.54
516.17	13 00	750	34.1	19.17	1.022	19.59	1.026	20.10	0.15	8 40	0.989	19.37
517.17	15 40	750	37.4	17.45	1.034	18.04	1.039	18.74	0.15	8 40	0.989	17.84
518.17	17 30	750	40.7	16.02	1.045	16.74	1.049	17.56	0.13	7 30	0.991	16.59
519.17	19 00	750	44.7	14.54	1.056	15.35	1.058	16.24	0.15	8 40	0.989	15.18
520.17	20 10	500	32.7	13.25	1.065	14.11	1.065	15.03	0.20	11 30	0.980	13.83
521.17	21 00	500	35.9	12.04	1.072	12.91	1.071	13.83	0.20	11 30	0.980	12.65
521.77						Water surface of nappe.						

of fact the Ott meters sometimes under-registered this component, obtaining the value, *O D,* which is less than *O B.* It was necessary, therefore, to correct some measured velocities by applying the small under-registration factors shown in Fig. 6. Thus, Column (7) in Table 2 is the product of Columns (5) and (6). The actual value of the current velocity, *O C,* was obtained by multiplying the horizontal component, *O B,* by the secant of the vertical angle, θ. The average value of this angle was read from Fig. 9 and was tabulated in Column (2). The component of the stream velocity which was perpendicular to the gauging section is *O A* (Column (13)), and was obtained by multiplying the horizontal velocity, *O B* (Column (7)), by the cosine of the horizontal angle, ϕ, the sine of which was observed at every point at which the velocity was measured.

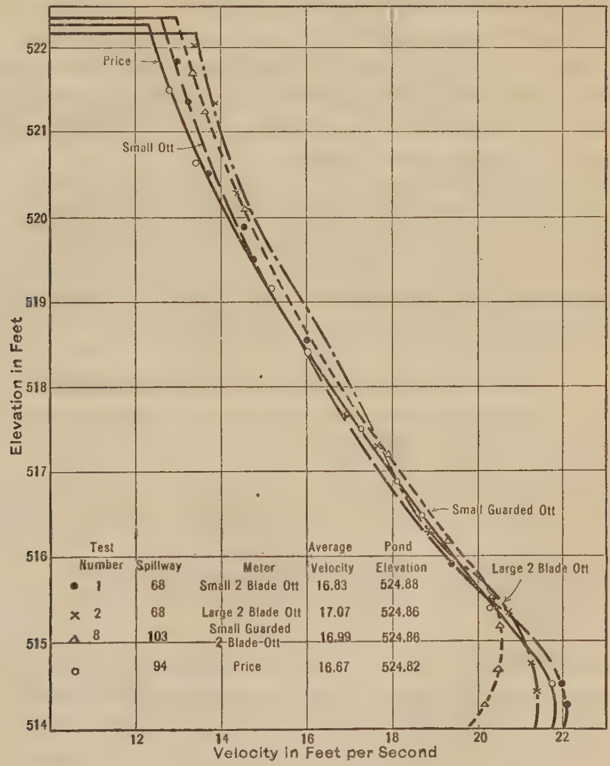


FIG. 11.—COMPARISON OF VERTICAL VELOCITY CURVES.

Computation of Spillway Discharge.—Table 2 is a sample showing the results obtained from the meter measurements in a vertical line 23 ft. from Pier No. 103 in Spillway No. 103 during Test 8. The results of all observations in each test were computed in this manner, and from such tables, curves were plotted showing the distribution of velocities in the different vertical lines. Such curves are shown in Fig. 11. The average velocity in each vertical was obtained by applying Simpson's rule to twenty equally spaced points on each vertical velocity curve. The values of mean velocity thus found were remarkably uniform across the spillway section. The maximum variation obtained, including tests with the most contracted nappes, was only 17 per cent.

When symmetrical conditions of flow existed there was little variation in depth of the nappe and current velocities in the middle half of the spillway, but the pier contractions caused a rapid decrease in depth at points closer to the face of a pier. It is chiefly for this reason that the distance between the vertical lines of measurement was decreased near the edge of the spillway.

The discharge of the spillway was computed from the data on average velocity and nappe depth in the manner commonly used in stream measurement; that is, by computing the average velocity, depth, area, and quantity of flow, in the nappe sections between verticals. A summary of the computed discharge for each test is given in Table 1, together with other essential data.

While the variations in pond level were relatively small and the average of that recorded at the east gauge checked very well with that recorded at the west gauge, pond level readings were weighted in proportion to the quantity of water being measured in obtaining the "Weighted Average Pond Level" given in Column (11) of Table 1.

The discharge coefficients given in Column (19), Table 1, verified the earlier belief that the coefficients used in the design of the dam were conservative by at least 10 per cent. Under normal operating conditions, the correct weir coefficients varied from minimum values close to 3.71 to maximum values of 3.90, depending on the character of the operation of the gates. Table 3 shows that the resulting coefficients are quite consistent.

TABLE 3.—COMPARISON OF WEIR COEFFICIENTS.

Method of spillway operation.	Coefficient.	Remarks
One spillway operating alone.....	3.71	Average of coefficients obtained in Tests 1 and 2 on Spillway No. 68 and Test 8 on Spillway No. 103.
Middle of three open spillways ...	3.78	Average of coefficients obtained in Tests 4 and 5 on Spillway No. 68.
Middle of seven open spillways...	3.90	Coefficient obtained in Test 7.
End gate of two open spillways ..	3.74	Coefficient obtained in Test 2.
End gate of five open spillways...	3.72	Coefficient obtained in Test 6.

Better approach conditions and reduction in the nappe contraction increase the discharging capacity of a spillway operating at the center of a larger group, while the end spillways of such a group have such severe contractions around the end pier that they discharge very little more water than a single spillway operating alone.

Contraction of the Nappe Around the Piers.—The piers reduced the discharge of water to a greater extent than that by which the area of unobstructed spillway was reduced by the concrete in the piers themselves. The weir coefficients in Table 4 were computed from the discharge flowing in the 8-ft. section remote from pier influence at the center of the nappe, in those tests in which the nappe was symmetrical. The reduction in discharge caused by the lateral contraction of the nappe as it passes around the piers with only one spillway open is more than twice that observed with many spillways open.

The form of the water surface of the nappe at the metering section which was observed in Tests 2, 3, 5, and 6, is shown in Figs. 12 and 13. Note the unsymmetrical form of the nappe surface in Tests 3 and 6 performed on a spillway at the end of a set of open spillways as compared with the more balanced surface curves in the other tests. There was less contraction in the nappe of Test 7 with many gates open than in Tests 2 and 8 on a single open spillway. The deeper pond near the power-house end of the dam seemed to produce a slight difference in the nappe contraction at the two sides of the spillway in Test 8.

TABLE 4.—VARIATIONS IN WEIR COEFFICIENTS.

Condition of operation.	Coefficient, C, in $Q = CLH^{\frac{3}{2}}$.		Percentage reduction in coefficient.
	At center of nappe.	Entire spillway.	
One spillway operating alone.....	3.92	3.71	5.4
Middle of three open spillways ...	3.92	3.78	3.6
Middle of seven open spillways...	4.00	3.90	2.5

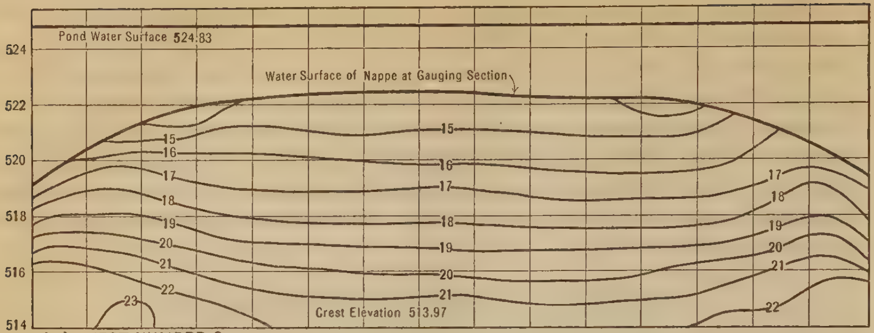
Distribution of Velocities Within the Nappe.—The contours in Figs. 12 and 13 and in similar curves drawn for all the tests show the actual velocity of the water which was measured throughout the nappe. The velocity shown is not the component perpendicular to the metering section, but the actual current velocity in the metering section regardless of direction of flow.

A study of these curves discloses the following significant facts with regard to the behavior of the water as it passes over the spillway:

> (1) Throughout the center of the spillway there is a remarkable uniformity in the velocity at a given depth below the pond level.
>
> (2) The velocities increase from a minimum at the water surface to maximum values which are only a few tenths of a foot above the crest, there being but relatively little retardation at this point due to friction.
>
> (3) Higher velocities than those existing in the center of the spillway are encountered a few feet from the pier faces where the nappe is formed by the water which is turning the bend around the pier nose. This increased velocity exists from the top to the bottom of the nappe as indicated by the elevation of the contours as the sides of the spillway are approached.
>
> (4) Those tests in which meters were operated close to the pier faces show a small stream of slowly moving water at the crest in the outside corners, indicating that the water followed with difficulty the contour of the spillway at these places.

In making comparisons between the several tests and various parts of the same spillway, cognizance should be taken of the fact that slight differences in the pond level which existed at the time of measurement in a given section may be responsible for small variations in contour position.

It is scarcely possible, from these figures based on slightly varying conditions, to note the significant facts disclosed in Fig. 14. In plotting this diagram the velocity readings taken at the middle of the spillway have been reduced to corresponding values such as would have existed with a uniform



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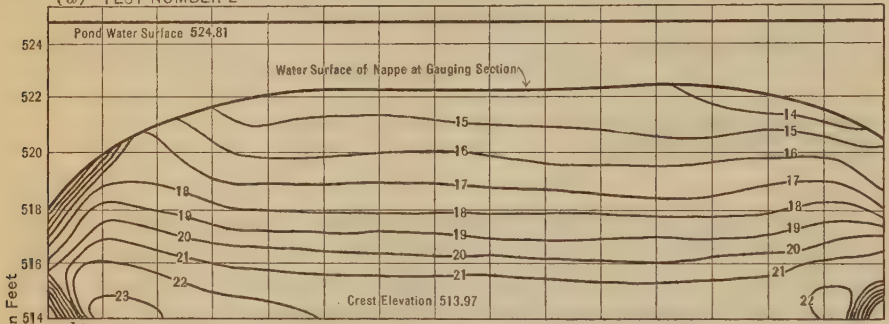
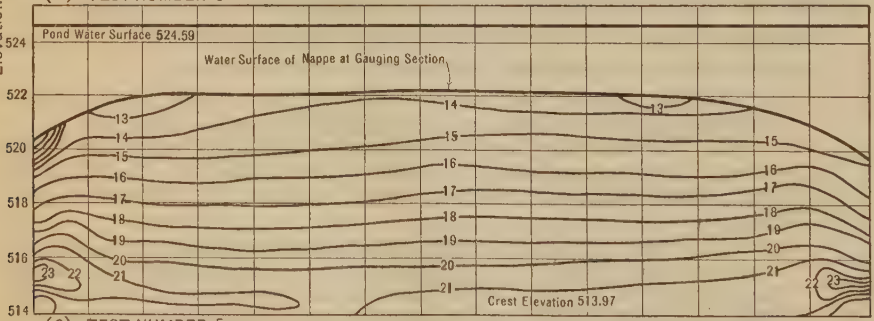
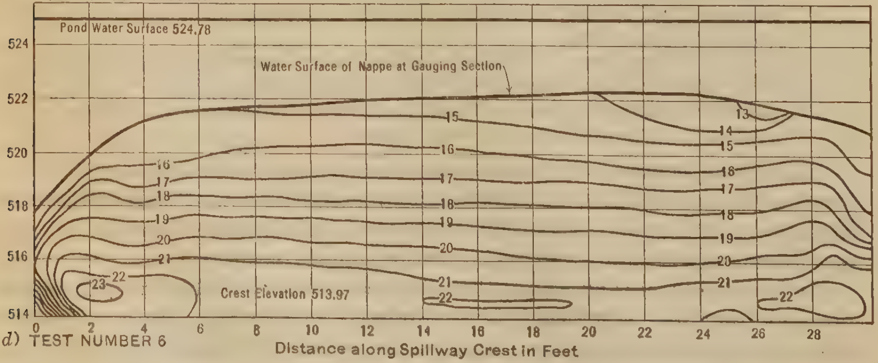


(b) TEST NUMBER 3



(c) TEST NUMBER 5



(d) TEST NUMBER 6

FIG. 12.—ACTUAL VELOCITY CONTOURS, IN FEET PER SECOND.

pond level at Elevation 524.80. A progressive increase in the number of open spillways seems to be accompanied by a progressive increase in the velocity and a depression of the nappe surface at the point of measurement. Thus, the larger spillway discharge coefficients with the greater number of open spillways may not be due entirely to the decrease in contraction of the nappe around the piers, but may also be produced by the higher velocities within the nappe itself when water is being drawn from a larger area in the pond, acquiring some initial velocity before reaching the spillway. This increase in velocity must be accompanied by a corresponding decrease in the pressure within the nappe. Considering the fact that the axis representing zero velocity is some distance to the left of the diagram, the difference in velocities is really of small consequence, hardly exceeding 5 per cent.

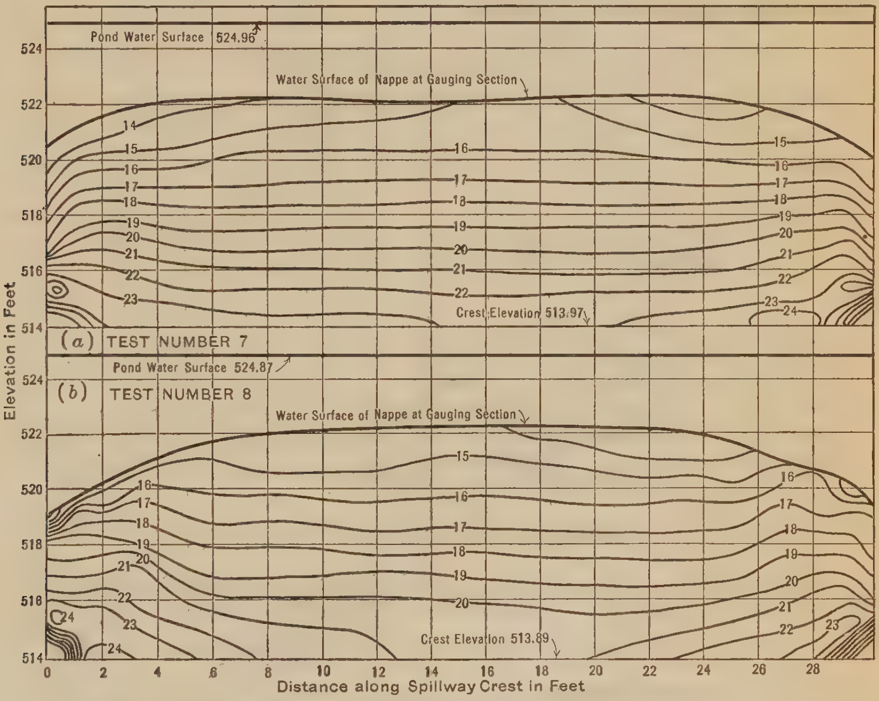


Fig. 13.—Actual Velocity Contours, Keokuk Spillway Calibration.

The curves of Fig. 14 indicate that the velocity at the surface of the nappe may exceed that acquired by the water in falling from the pond level to this point. This excess, although small indeed, was observed in many instances and may be created by the friction of the lower filaments of the nappe upon the slower moving filaments at the surface. It is possible, however, that this phenomenon may have been caused by an initial wind surface current, or a tendency on the part of the meter to over-register near the water surface, although observations generally were not made within 6 in. of the surface.

The higher nappe velocities near the piers result from lower pressures in the interior of the nappe, but even at these places the maximum nappe velocities did not attain the limit which would exist if the interior nappe pressure were reduced to zero.

Condition of Stream Bed Up Stream from Spillways.—Soundings were made to determine the elevation of the bed of the river within a distance of 25 ft. from the spillways upon which tests were made. These soundings were made with a steel tape from the platform of the traveling crane. Contours of the bed of the pond are shown in Fig. 15 and profiles at certain sections are shown in connection with Fig. 3. There is evidence of sedimentation on top of the irregular deposit of construction materials ahead of most of the spillways, although all this material was at least 7 ft. below the crest of the spillways under investigation. The nature of the stream bed ahead of Spillway No. 103 was obviously responsible for the slightly unbalanced form of the nappe cross-section in Test 8.

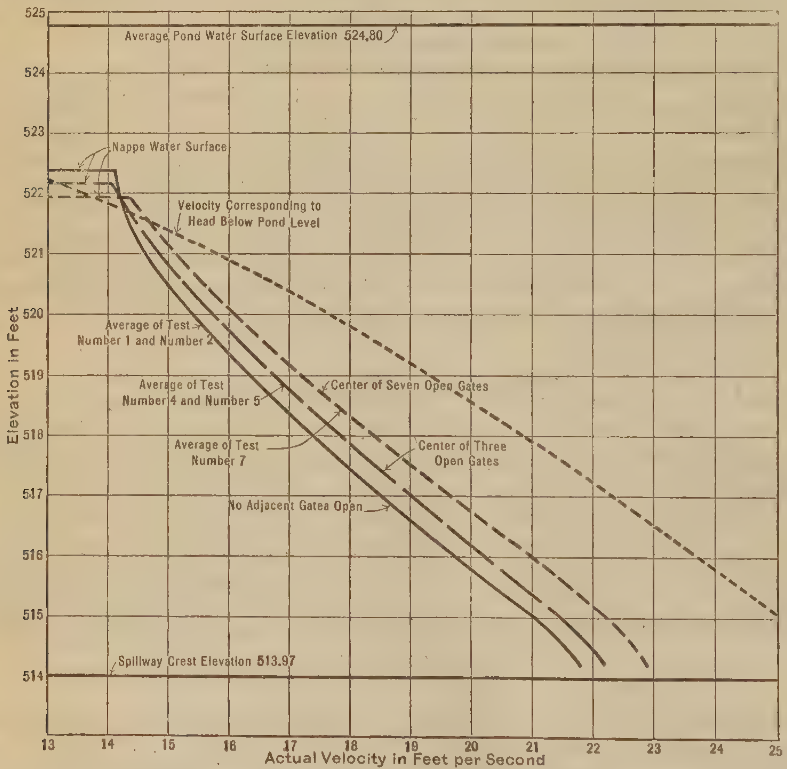
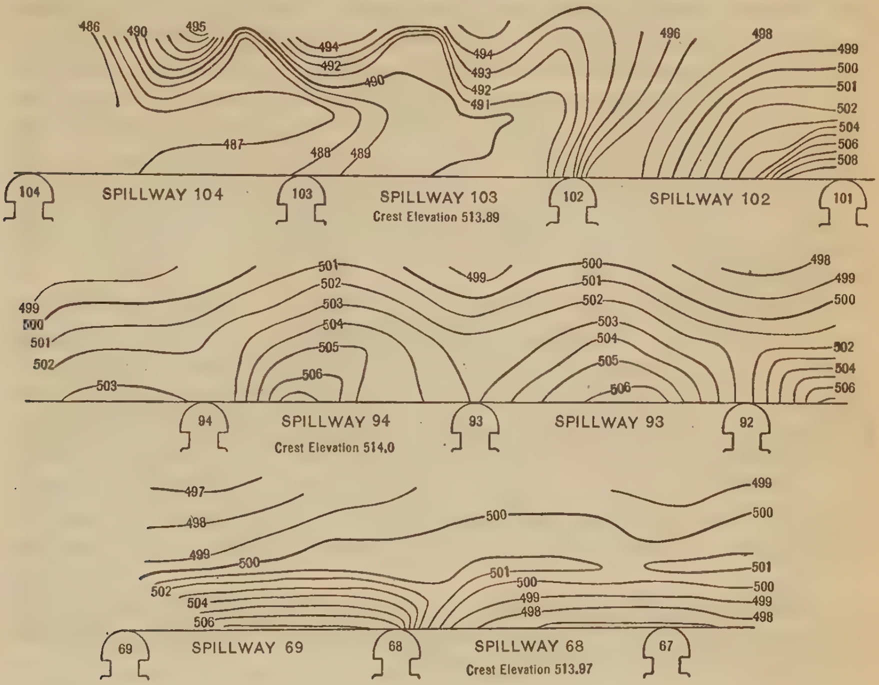


FIG. 14.—CURVES OF ACTUAL VELOCITY, STATION 15, KEOKUK SPILLWAY CALIBRATION.

Surface Drop Near Open Spillway.—Water surface elevations were taken in the immediate vicinity of Spillway No. 94 when discharging alone with no closely adjacent spillways open, in order to show accurately the shape of the depression of the water surface within 25 ft. of an open spillway. The result is shown by the contours in Fig. 16. It is interesting to note the change from the convex to concave contour as the nappe passes over the spillway. This work was done when the lake was very calm. A tape with a heavy sounding weight attached at the zero end was lowered from the platform of the traveling crane, and readings of the tape were taken with a level set up on the dam. Observations which were correct within 0.02 ft. were probably obtained in this manner.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502
504
506
69
SPILLWAY 69
68
SPILLWAY 68
Crest Elevation 513.97
67

FIG. 15.—CONTOURS OF BOTTOM OF POND.

Accuracy of Work.—The agreement of the various tests which were repeated under differing conditions is one indication of the relative accuracy of the discharge measurements.

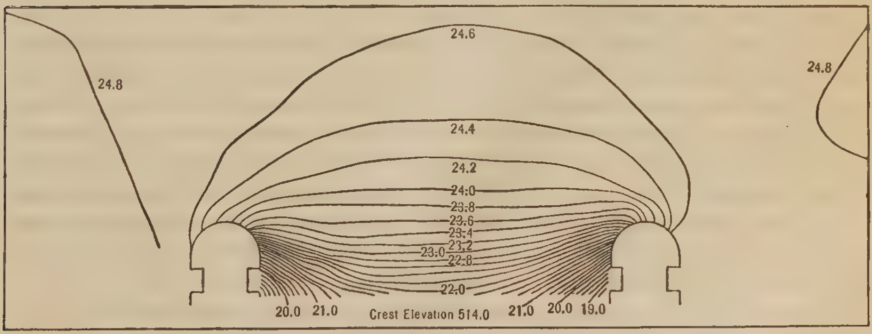
24.8
24.6
24.8
24.4
24.2
24.0
23.8
23.6
23.4
23.2
23.0
22.8
22.0
20.0
21.0
Crest Elevation 514.0
21.0
20.0
19.0

FIG. 16.—WATER SURFACE CONTOURS, SPILLWAY NO. 94, KEOKUK DAM.

Although Test 1 was regarded from the outset as a preliminary run, resulting in the more or less awkward and experimental handling of the equipment, its result departs 2.1% from Test 2 performed on the same gate with the same meter, and under the same conditions. Test 8 differs only 0.8% from Test 2, although it was performed on a different spillway with a different meter, but under similar operating conditions. Test 5, a repetition of Test 4, performed with different meters, gave a result agreeing with Test 4 within 0.1 per cent.

Mention has already been made of the ideal stream-line flow existing at the metering section which enabled the repetition of velocity readings at a given point with almost exact duplication. In six instances the observations of the velocities existing in certain vertical lines were repeated with the same spillway operating conditions, after hours or days of intermission. In no case did the point velocities at a given level differ from those previously obtained by more than 5%, nor in most cases more than 2 per cent. The agreement of the average of the velocities in the vertical lines was much closer than this.

Extreme possibilities of variation are presented in Fig. 11, which shows vertical velocity curves taken at the middle station with four different meters on three different spillways when they were discharging independently of others. It is not to be expected that the curves for different spillways with differing depths of approach and crest conditions have exactly the same form; but, considering the location of the axis of zero velocity, it must be admitted that the difference in the curves is remarkably small. The maximum variation in the average vertical velocity obtained from these curves is 2.3%, some of which may be accounted for by the difference in lake level in these two instances.

A consideration of every possible source of error indicates that the discharges computed from the data obtained in these tests may be considered reliable within a possible error of 2 per cent.

Experiments on a Model of the Keokuk Spillways

The possibility of determining the proper discharge coefficients for the Keokuk spillways by means of small-scale experiments on a model of the dam was suggested before the field experiments at Keokuk were started. The model tests involved only one-sixth the expense of the full-scale experiments, and studies under varying heads could be made easily on the model, while operating conditions at Keokuk required the maintenance of a constant pond level. In the minds of many, however, such small-scale tests are unconvincing. In order, therefore, to test the reliability of model experiments it was decided at the outset to attempt the calibration of the Keokuk spillway by measurements made both on the actual spillways and on a model of them. The canals of the Hydraulic Laboratory of the State University of Iowa were not available for this work immediately after the completion of the measurements at Keokuk; hence the model tests were not performed until September and October of the following year.

Description of the Model.—A model of three spillway sections of the Keokuk Dam was constructed of wood on a scale of one-eleventh the size of the actual dam, a section of which is shown in Fig. 3. Great care was taken in duplicating exactly to scale the form of the crest, and the shape and spacing of the piers. The wood which was chiefly redwood, was planed and sandpapered to a smooth finish for use in the early experiments, although in later tests it was given, successively, a smoother finish with shellac and a rougher finish with sand.

The model was erected 80 ft. from the entrance to the 10 by 10-ft. concrete testing canal of the laboratory. The three spillways with two full piers and two half piers at the canal sides occupied the full width of the canal exactly. The water for the canal was supplied from the pond on the Iowa River above the University Dam. The river furnished an unlimited quantity of water for these experiments (an average flow of 1 500 cu. ft. per sec.). Baffles in the canal a few feet down stream from the head-gates thoroughly diffused the stream filaments before they passed down the channel toward the model. Leaving the model the water entered a basin and, after passing through baffles, it had 50 ft. of unobstructed channel in approaching a suppressed, sharp-crested, rectangular weir at the down-stream end. The weir and basin were 10 ft. wide and 7 ft. deep below the crest.

Views of the up-stream and down-stream faces of the model are shown in Figs. 17 and 18. The spillway sections are 2.727 ft. wide, corresponding to the 30-ft. width of the spillway at Keokuk. Water passing over the model spillways at a depth of 1 ft. duplicated the normal condition at Keokuk with a lake elevation of 525 ft., 11 ft. above the spillway crest.

Each spillway of the model was provided with a gate, so that experiments could be made upon the dam with any combination of the three spillways in operation.

Nature of the Experiments on the Model.—The primary objective in the model experiments was to obtain values of discharge coefficients under conditions similar to those which existed during the Keokuk measurements and, by varying the water level above the model, to determine values for these coefficients under abnormal pond-level conditions, such as existed during the early years of operation at Keokuk.

During each experiment the water level above the model was carefully observed by taking ten to twenty readings on a hook-gauge set so as to read the level in a stilling-well connected to the canal about 10 ft. up stream from the model. Likewise, the quantity of water passing over the model during each test was measured by observing the head on the weir as indicated by a hook-gauge operating over a stilling-well connected to the weir channel 16 ft. up stream from the weir. Ten to twenty readings were secured during each run, and their average was used in the Bazin weir formula in computing the rate of flow. Careful measurements were made and levels taken daily throughout the period of experimentation on both the weir and model in order to detect the slight variations which change in temperature and moisture conditions produce on such apparatus.

The model was tested with all three gates in operation through a range in water level above the crest which varied from 0.4 ft. to 1.5 ft., corresponding to a variation from 4.4 to 16.5 ft. at Keokuk. Other discharge experiments were conducted when only single gates were opened with the spillway discharging under a head close to the normal head of 1 ft.

For these tests the model operated under the following conditions:

1.—Planed wood surface on model duplicating Fig. 2 with no deposit of silt against the dam.

2.—Planed wood surface on model with a deposit of sand against the dam duplicating the field conditions ahead of Spillway No. 68 as shown by the contours in Fig. 15.

3.—Shellac finish on the model with the deposit ahead of the dam.

4.—Sand finish on model with the deposit ahead of the dam.

In addition to the four series of discharge experiments, six other tests were made to determine the velocity distribution within the nappe when the model was being operated under conditions similar to those existing during the Keokuk measurements. A small Ott current meter and a Pitot tube were used for this purpose.

In order to compare the shape of the nappe flowing over the model spillway with that passing over the Keokuk Dam, during eight of the tests, measurements of the surface of the nappe were obtained across the section corresponding to that in which the meters were operated at Keokuk.

Experiments on Effect of Condition of Model on Discharge.—In building a model of the concrete spillways at Keokuk, the actual roughness of the surface probably should be scaled down according to some ratio as well as the dam itself. Since the surface of the concrete in the full-sized dam appeared to be remarkably regular for this class of work, it was thought that the surface of the model should be at least as smooth as wood could be planed. Accordingly, the first series of experiments were performed on the planed and sand-papered model as constructed in the canal with no surfacing of any kind. The resulting coefficients when the water was passed over all three spillways are shown by the points in Fig. 19 which define Curve 1.

A deposit of sand was next placed above the dam in an attempt to duplicate that existing above the Keokuk Spillways Nos. 67, 68, and 69, where most of the field measurements were made. Small stakes were cut to proper length and fastened at points corresponding to those places where the soundings had been taken for the preparation of Fig. 15. Wet sand was then packed against the dam and filled in between the stakes forming the most natural surface between the tops of the sticks, leaving them barely exposed. The sand was further compacted by inundation in passing water over the dam, and its surface remodeled until it was firm enough so that but little alteration of its contour was caused by further inundation or experimentation. The results from this series of experiments are shown by points defining the position of Curve 2 in Fig. 19.

For all heads on the model above 0.63 ft. the deposit of silt seemed to have caused a lowering in the discharge of the three spillways, although at a head of 1.00 ft. corresponding to the normal head of 11 ft. at Keokuk there was less than 0.6% more water passing over the model without the silt deposit than passed over with it.

The small amount of alteration in the coefficient caused by the deposit that now exists ahead of the Keokuk Dam was also observed in the Keokuk

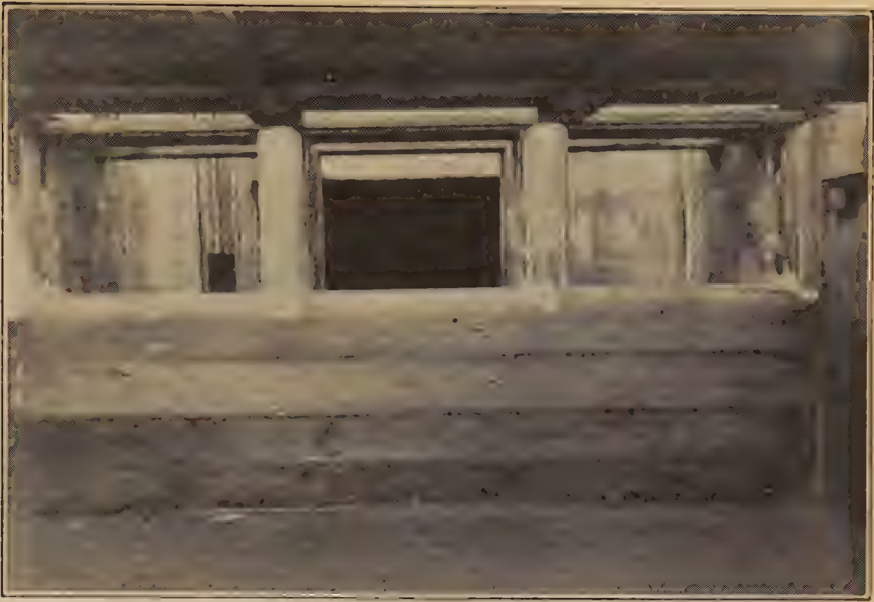

FIG. 17.—UP-STREAM FACE OF MODEL OF THREE SECTIONS OF THE KEOKUK SPILLWAY.

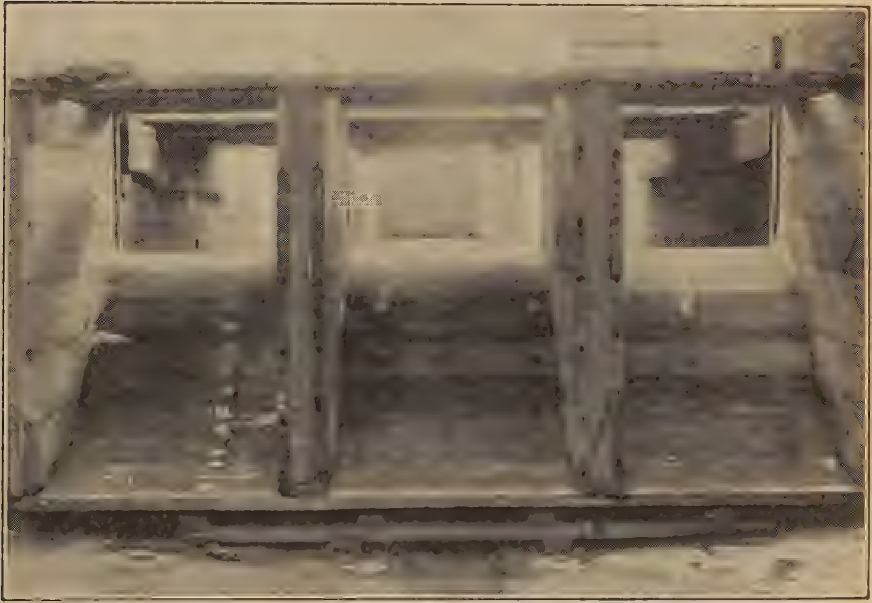

FIG. 18.—DOWN-STREAM FACE OF MODEL OF THREE SECTIONS OF THE KEOKUK SPILLWAY.

measurements, in which the results for Spillway No. 103 were in substantial agreement with the average of those obtained on Spillway No. 68, although the pond was 6 to 8 ft. shallower ahead of the latter spillway.

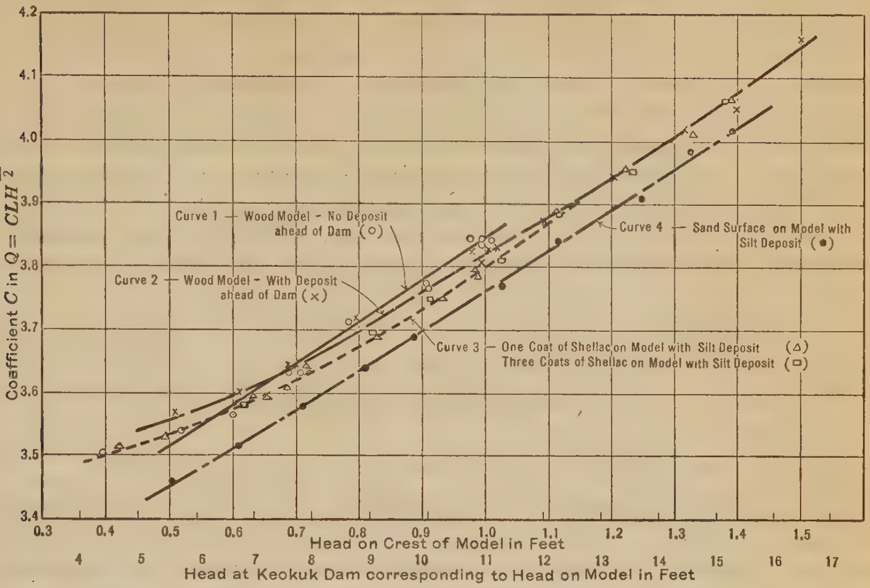

FIG. 19.—DISCHARGE COEFFICIENTS FOR MODEL OF KEOKUK SPILLWAY.

A series of experiments was next performed after one coat of shellac had been applied to all wooden parts of the spillway which came in contact with the water. This shellac was not applied until after the wood had thoroughly dried. Then, suspecting that one coat of shellac might not have produced the smoothest possible surface, two additional coats were applied, and another series of experiments were performed. There appeared to be no measurable difference in the results after the application of one or three coats of shellac, and the application of the shellac itself seemed to alter the discharge relatively little, as shown by Curve 3 in Fig. 19. When water was passing over the model with a head of 1 ft., the discharge seemed to be decreased about 0.4%, although at the greater heads the difference could not be detected. It is perhaps surprising that the smoother surface of the shellac caused a diminution of the discharge. However, this same effect of shellac has been noted in experimentation with other hydraulic apparatus, and it appears that water passes with less friction over a well-smoothed wooden surface (which soon becomes slippery under water) than over a shellac surface to which there may be some molecular adherence.

The shellac surface of the model was next treated with a fresh coat of shellac on which sand (passing a No. 28 Standard Tyler sieve, but held on the No. 48 sieve) was thrown until the entire surface was thoroughly covered. The result of the series of experiments performed on the model in this condition is indicated by Curve 4 in Fig. 19. Except for the lower heads the increased friction of the sand surface caused a diminution of the discharge amounting to approximately 1 per cent. However, when the head was small the difference was increased.

It was rather surprising that these various surface treatments produced so little difference in the resulting coefficients, and it indicates what degree of confidence may be placed in the results from models of the size used in these tests when the value of the surface friction of the model is in question.

It is quite certain that the sand produced a surface on the model which had greater relative roughness than that of the concrete on the Keokuk spillways; and it is believed that the results from the natural wood surface with the silt deposit ahead of the model may be taken as typical of that existing at Keokuk. The following comparisons may then be made between the coefficients obtained at Keokuk and those obtained under like conditions on the model:

		Coefficient.
At Keokuk:		
Tests 4 and 5	Center of three open spillways	3.78
Test 7	Center of seven open spillways	3.90
Model tests:		
Average of three open spillways		3.82

The coefficient obtained from the model seems to be in good agreement with the large-scale results at Keokuk. It is not as large as that obtained in Test 7, since it represents the average of three spillways, the velocities approaching the two outside ones being affected by the friction on the sides of the testing canal. It is not as small as that obtained in Tests 4 and 5, since it probably represents the operation of a larger number of spillways, with all extraordinary contraction suppressed.

Experiments on the Discharge of Only One Spillway.—Since the discharge as computed over a sharp-crested weir has less certainty under low heads, and that low-head limit for these tests had been arbitrarily set at 0.4 ft., it was impossible by passing the water over the 10-ft. weir to measure accurately the discharge over a single spillway which was less than 3 ft. long when the pond level was less than 0.9 ft. above the crest of the model. Accordingly, most of the tests on the single spillways were performed under a head of about 1.0 ft. which duplicated the normal conditions existing at Keokuk. However, coefficients for the middle spillway of the three were obtained throughout a small variation in head. The discharge coefficients which were obtained are given in Table 5.

The results are in agreement with those obtained when three spillways were operated with regard to the relatively small effect which was produced by the deposit of silt ahead of the dam and the application of the shellac to the surface of the model.

The coefficients are lower than those obtained with the three spillways in operation, due to the extraordinary contraction existing around the piers when the single spillways draw their discharge from the entire width of the channel. The coefficients obtained from the two end spillways are identical as they should be with accurately constructed apparatus; and the discharge through the center spillway is less than that through either of the end spillways due to the greater contraction of the nappe around both piers in this case. Neglecting the effect of friction along the sides of the canal, the discharge over the end spillway is similar to that which would pass through one of only two adjacent spillways in operation in a pond with more spillways than were provided by the present model.

TABLE 5.—DISCHARGE COEFFICIENTS, C, IN $Q = CLH^{\frac{3}{2}}$

Spillway.	WOOD MODEL.			WOOD MODEL WITH SILT DEPOSIT			SHELLAC.		
	No. of experiment.	Average head, in feet.	Average coefficient.	No. of experiment.	Average head, in feet.	Average coefficient.	No. of experiment.	Average head, in feet.	Average coefficient.
East end...	4	0.973	3.77	2	0.980	3.77	1	0.984	3.69
Middle.....	5	0.995	3.69	4	1.025	3.69	1	0.985	3.69
West end..	..	......	......	1	0.996	3.77	..	......	......

The results obtained from these experiments on the model agree within 0.8% with the coefficients obtained under similar conditions at Keokuk:

		Coefficient.
Keokuk Tests 1, 2, and 8...	Only one spillway..........	3.71
Model tests................	Center spillway............	3.69
Keokuk Test 3............	End of two open spillways...	3.74
Model test.................	End spillway...............	3.77

Variation in Coefficient with Change in Head.—Fig. 19 shows considerable variation in the discharge coefficient with an increase or decrease in head from normal conditions. Experiments performed when water was discharging over only the middle spillway gave smaller coefficients increasing less rapidly as the head increases. The resulting curve (Fig. 20) seems to indicate that under low heads there is less difference in the coefficient when many spillways are in operation compared with that when a single spillway is open.

Coefficients which exceed 4.0 are not usually encountered, but it is evident that they may be secured when water discharges at great depths over well-designed spillways.*

That the coefficient would decrease rapidly for lower heads on the spillway was anticipated from previous measurements on the Keokuk spillway at low pond levels and from gaugings of the Mississippi River during certain floods when low lake levels existed. These other measurements (Table 6), while not comparable in accuracy with the work described in this paper, indicated that the discharge coefficient of the Keokuk spillway decreased to a value close to 3.0 when the nappe passed over the spillway at a shallow depth, duplicating the phenomena of flow over a broad-crested weir with a rounded approach. A comparison of the results from the model experiments with all measurements made at Keokuk is shown in Fig. 20.

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* *Water Supply and Irrigation Paper No. 200,* Pl. XXXVI, U. S. Geological Survey; Report of Hydraulic Power Committee, National Elec. Light Assoc., 1923.

The model experiments are in good agreement with the 1924 measurements at Keokuk and are in as close agreement with the measurements listed in Table 6 as one could expect from the nature of these other determinations of the discharge coefficient.

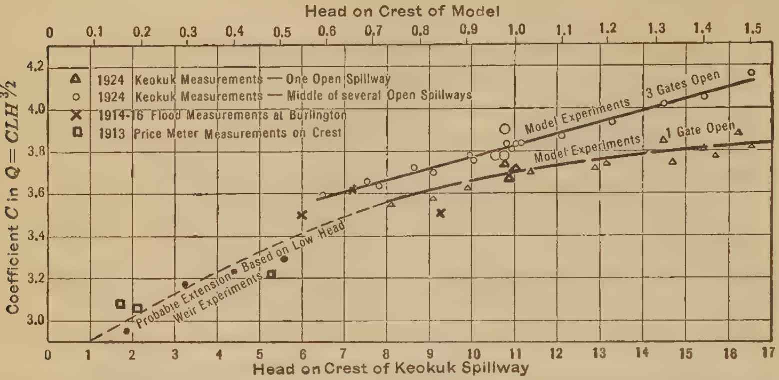

FIG. 20.—SUMMARY OF MEASUREMENTS OF DISCHARGE OVER KEOKUK SPILLWAY.

Measurements on the Form of the Nappe.—In order to compare the form of the nappe passing over the model with that measured at the metering section at Keokuk, this exact section was located on the model and, by means of a pointed rod, about twenty readings to the water surface were taken to the nearest hundredth of a foot at varying intervals across the nappe.

TABLE 6.—MEASUREMENTS OF DISCHARGE OVER KEOKUK SPILLWAYS BEFORE 1924.

Date.	Pond elevation, in feet.	Head, in feet.	Discharge per spillway, in cubic feet per second.	Coefficient in $Q = CLH^{\frac{3}{2}}$.	Remarks.
June 5, 1913....	515.63	1.71	206.3	3.06	By R. H. Bolster with Price meter on crest.
June 6, 1913....	515.93	2.01	260.1	3.04	By R. H. Bolster with Price meter on crest.
June 11, 1913...	519.20	5.28	1 165.8	3.21	By R. H. Bolster with Price meter on crest.
June 22, 1914...	521.22	7.22	2 104	3.61	Flood flow through 41.4 gates computed from current-meter measurement at Burlington, Iowa.
June 10, 1915...	520.00	6.00	1 545	3.51	Flood flow through 64 gates computed from current-meter measurement at Burlington, Iowa.
March 31, 1916.	523.22	9.22	2 942	3.50	Flood flow through 55.1 gates computed from current-meter measurement at Burlington, Iowa.

The nappe surface was measured in this manner when the model with the planed wooden surface was operating with the sand deposit against the dam and also without that deposit or sediment. Under each of these conditions, measurements were made on the nappe of the center spillway when operating alone, with all three spillways open, and with only one of the other spillways in operation. Readings were also taken to the surface of the nappe of one of the end spillways when operating alone.

A comparison of the results of the measurements on the model with the deposit of silt ahead of the dam and those taken on the Keokuk spillway under similar conditions is shown in Fig. 21. The model tests shown were conducted under the following circumstances: Test 45 was on Spillway No. 2, with Spillways Nos. 1 and 3 open; Test 46 on Spillway No. 2, with Spillway No. 1 open; and Test 47 on Spillway No. 2 and Test 48 on Spillway No. 1, with no other spillways open. The corresponding data for the field tests are given in Table 1. The depths of water in the nappe which were observed at Keokuk have been divided by the scale ratio of 11 in 1, in order that they might be plotted on these diagrams for comparison. In all cases the model seemed to represent quite faithfully the form of the larger nappe. In Fig. 21(*c*) the difference in average pond level between that existing at the time the model measurements were made and that existing during Test 5 at Keokuk, fully accounts for the separation of the two curves.

The four measurements of the nappe which were made when there was no sand deposit ahead of the dam gave water surface curves very similar to those shown in Fig. 21, except that the surface was lifted between 0.01 and 0.02 ft. uniformly across the entire spillway as if the deeper water ahead of the dam resulted in a very slight thickening of the nappe. A difference of this sort between Spillways Nos. 103 and 68 located in shallower water, was not noted in the tests at Keokuk.

In order to test the accuracy of using the middle third of the Keokuk spillways as an index of what flow would be obtained over an unobstructed crest, the piers were removed from the model spillway and an accurate determination was made of the cross-section of the nappe. Fig. 21 (*d*) gives an interesting comparison between the nappe cross-section at the location of the middle spillway of the unobstructed crest with that observed when one and three spillways were open and the piers were in place. The observations for Test 47 have been reduced to the same pond level of 1 ft. above the crest, which existed in the other two tests. It will be noted that the cross-section of the nappe in the center section of all the tests is practically identical, showing clearly that the flow in the center section of the spillway flanked by piers can be used for a reliable determination of the flow over an unobstructed crest. It is believed that this one experiment proves beyond a doubt the correctness of the analysis which resulted in the coefficient of 4.00 for the normal head on the unobstructed crest at Keokuk.

The amount of contraction of the nappe caused by the piers is very evident from Fig. (21(*d*) and the fact that it decreases with the opening of adjacent spillways is quite significant. From this diagram it proved entirely possible to compute within 1% the value of the discharge coefficients previously obtained by flow measurements for the two cases—of a spillway discharging by itself and at the center of a group. The computation consisted

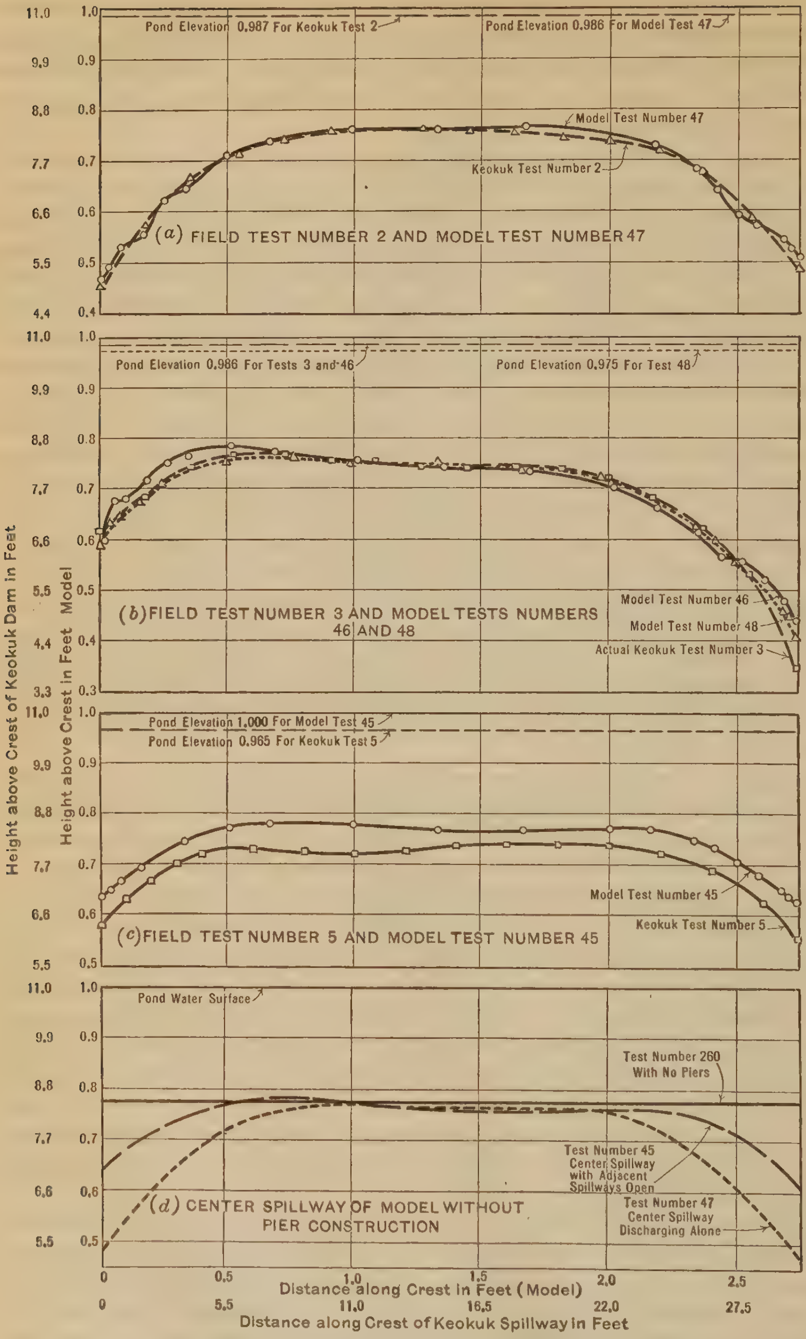

FIG. 21.—COMPARISON OF NAPPE CROSS-SECTIONS.

of subtracting from the quantity of water passing over an unobstructed spillway that discharge which would have passed through the area above the actual nappe section and the level water surface of Test 260. The elements of the computation were as listed in Table 7.

TABLE 7.—VALUES USED IN COMPUTATIONS.

Item.	No piers or obstructions.	Single spillway open.	Center of several open spillways.
Area of nappe, in square feet	2.124	1.940	2.072
Reduction by piers, in square feet	0	0.184	0.072
Percentage of reduction in area	0	8.7	3.4
Ratio of average velocity in reduced portion to average velocity in full nappe	....	0.78	0.76
Reduction in discharge by piers, in percentage	0	6.7	2.6
Discharge coefficients in model experiments	3.91	3.69	3.82
Discharge coefficient computed from reduction of nappe by piers	3.91	3.65	3.81

It is interesting to note the close agreement between the values of reduction in coefficient given in Table 7 and that computed for the Keokuk measurements in Table 3.

The form of the nappe at the crest of the spillway changed considerably when the head was raised above or lowered below the normal condition. The views in Figs. 22 and 23 with only the center spillway in operation show that the piers cause relatively little reduction of the nappe section when the head is low, but this increases to a very large amount when the head is 50% greater than normal. The increase in the magnitude of the pier obstruction with increasing head is probably responsible for the small increase in the discharge coefficient at high heads when a single gate is open as shown in Fig. 20.

Measurements of Velocities in the Nappe Flowing Over the Model.—A small Ott current meter (No. 2914) was first used in an attempt to measure the velocities existing in the center spillway of the model at points in the nappe which were in the section corresponding to that in which the meters were operated at Keokuk. Only two complete measurements throughout the nappe were made with this instrument before it was abandoned in favor of a small Pitot tube made from ⅛-in. brass tubing. The meter had a propeller 0.18 ft. in diameter and, when mounted on a ¾-in. rod, caused no apparent obstruction to the flow, since the metering section was some distance down the falling nappe. The diameter of the wheel of the meter, however, was almost one-fourth the depth of the nappe, and hence many points in a vertical section could not be taken with this instrument.

The Pitot tube had been previously manufactured and calibrated for use in measuring velocities in the nappe of a sharp-crested weir and proved a very excellent instrument for this kind of work. The use of a tube of this kind in the nappe requires watchfulness in order to prevent the entraining of air when operating near the water surface where the tube is likely to split the nappe and cause a crevice of air behind it. The behavior of the tube when water approaches it at an oblique angle, as it does in the upper sections and sides of the nappe, is the main element of uncertainty involved in its use, but coefficients under these conditions may be obtained by calibration. There was a striking resemblance between the velocity distribution as observed in the model and that obtained at Keokuk under similar conditions, revealing all the characteristics previously mentioned.

In each test the total discharge of the spillway was calculated by treating these velocity measurements in the same manner as that by which the measurements at Keokuk were computed. This afforded a sort of check on the weir measurements. A summary of these results is given in Table 8.

TABLE 8.—SUMMARY OF DISCHARGE MEASUREMENTS ON CREST OF MODEL.

Test No. (1)	Other spillways operating. (2)	Instrument used. (3)	Number of verticals. (4)	Number of points in vertical. (5)	Discharge by velocity reading. (6)	Discharge by weir. (7)	Percentage of difference. (8)
31	None	Meter	8	6	9.67	9.74	—0.9
68	None	Meter	8	7	10.13	9.93	2.0
69	None	Pitot tube	13	9	9.72	9.77	—0.5
84	All	"	14	9	11.15	10.89	2.4
85	No. 3	"	13	9	10.06	10.14	—0.8

It is to be noted that in Column (7) of Table 8, Test 84 is the average discharge of all three spillways which is probably less than that flowing over the center spillway measured by the Pitot tube. In Test 85 it is the average of the center and end spillways which is more than the quantity passing over the center spillway alone which was measured by the Pitot tube.

EFFECT OF FORM OF PIERS ON DISCHARGE OVER SPILLWAY

As a conclusion to the tests on the model the effect produced by three other pier forms was investigated in order to compare the behavior under these conditions with that for the model of the Keokuk spillway. These experiments were performed in November and December, 1925, and in May, 1926.

Description of Pier Forms.—The additional three types on which experiments were made are shown in Fig. 24 and are designated as Forms *B*, *C*, and *D*. Form *A* is the pier nose of the Keokuk model. The new piers were made of wood and were duplicated on the two full piers and two half piers of the model. Neither the form of the crest of the dam nor the silt deposit ahead of the dam was altered for these tests, but the sand finish which had been placed on the dam during the previous experiments with the Keokuk model was removed. Piers *B* and *C* were formed by extending the piers up stream from the dam in order to produce the lateral contraction of the nappe before it passed over the crest.

Pier *D* was constructed with a circular nose at the up-stream end of the Keokuk piers, but was formed with only one-half their radius. The further increase to full pier width was made gradually, thus producing a large part

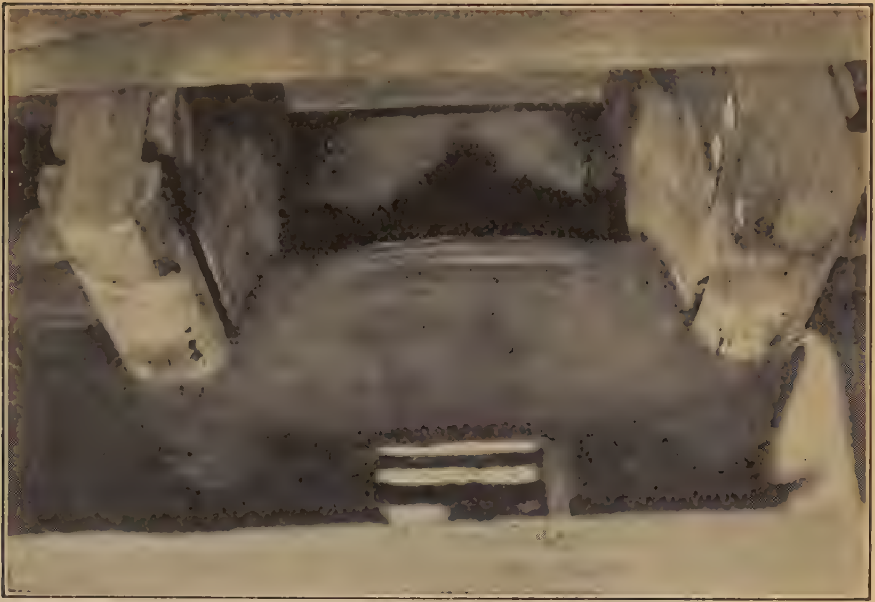

FIG. 22.—WATER DISCHARGING OVER CENTER SPILLWAY OF KEOKUK MODEL UNDER A HEAD OF 0.6 FEET.

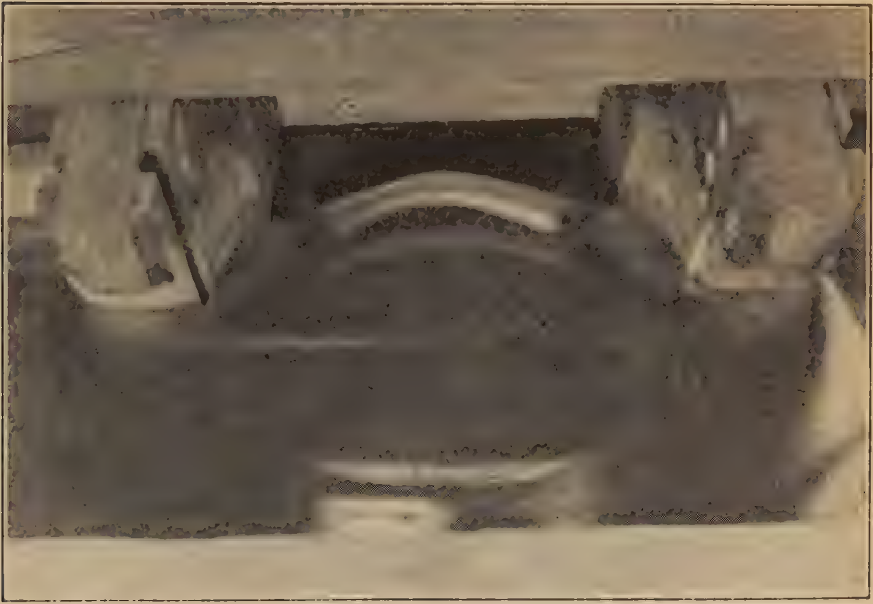

FIG. 23.—WATER DISCHARGING OVER CENTER SPILLWAY OF KEOKUK MODEL UNDER A HEAD OF 1.5 FEET.

of the lateral contraction of the nappe as it was falling over the crest of the dam. There was no provision for a gate slot in Pier *D*.

Experiments and Their Results.—The discharge over all three spillways and the several spillways operating singly was measured in the same manner as in the experiments on the Keokuk model. Finally, all the piers were removed and discharge measurements were made on the spillway, free from all obstructions, occupying the full width of the testing canal. The resulting coefficients when all three spillways were in operation are shown in Fig. 25 (*a*).

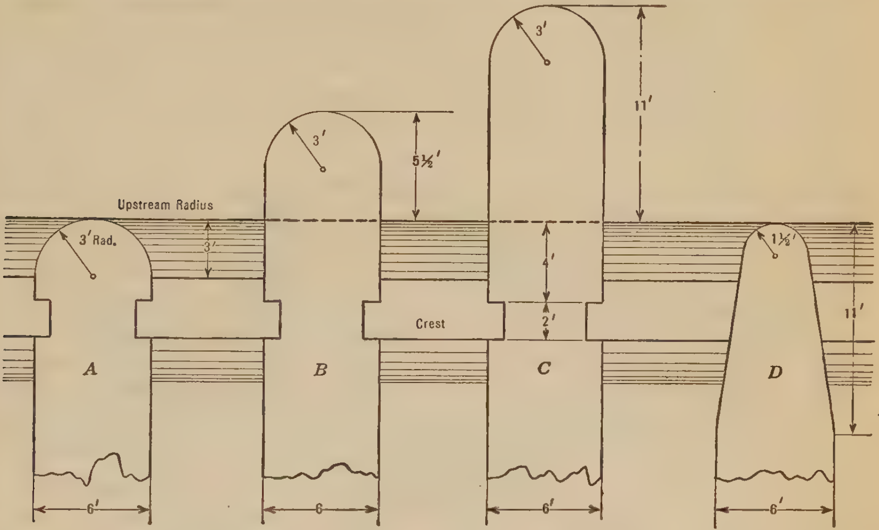

FIG. 24.—TYPES OF SPILLWAY PIER NOSES.

Pier *B*, the nose of which extended up stream from the face of the model dam 6 in. (or 5.5 ft. in the actual dam), gave higher coefficients than the Keokuk model when the head was less than 1.22 ft. In fact, it allowed a higher discharge through the spillway than Pier *C*, with a nose extending 12 in. up stream from the dam, for heads lower than 0.95 ft., which is close to the normal head at Keokuk. Because the coefficients obtained with Pier *B* seemed exceptionally large throughout the lower heads, the entire series of tests on this pier was repeated after a lapse of one week. The results confirmed those of the previous series in every respect. At heads between 0.95 ft. and 1.4 ft. Pier *C* allowed more water to pass than Pier *B*, but the maximum difference amounting to only 1%, is of little significance.

The *D* type of piers allowed considerably more water to pass than any of the other forms as shown by the coefficients in Fig. 25 (*a*), which were computed on the basis of the same net crest length between piers. About 8% more water could be passed over the Keokuk spillway provided the piers were shaped in this manner. It may not have been entirely consistent to compute the coefficients for Pier *D* on the basis of the crest length of the other models, since the flat part of the crest at its up-stream edge measured 2.953 ft. instead of 2.727 ft., which is the width between the piers after the water has passed down the face of the dam. Had the coefficients for Pier *D* been computed on the basis of a crest length of 2.95 ft., they would have been very similar to those obtained with the Keokuk model throughout the entire range in head.

A series of experiments on the spillway with all piers removed gave coefficients which were higher than those obtained with Piers *A*, *B*, and *C* for all heads greater than 0.94 ft. It is interesting to note that below this head there is some indication that it is possible for a dam with piers to have a higher discharge coefficient per linear foot of open spillway than one without these obstructions.

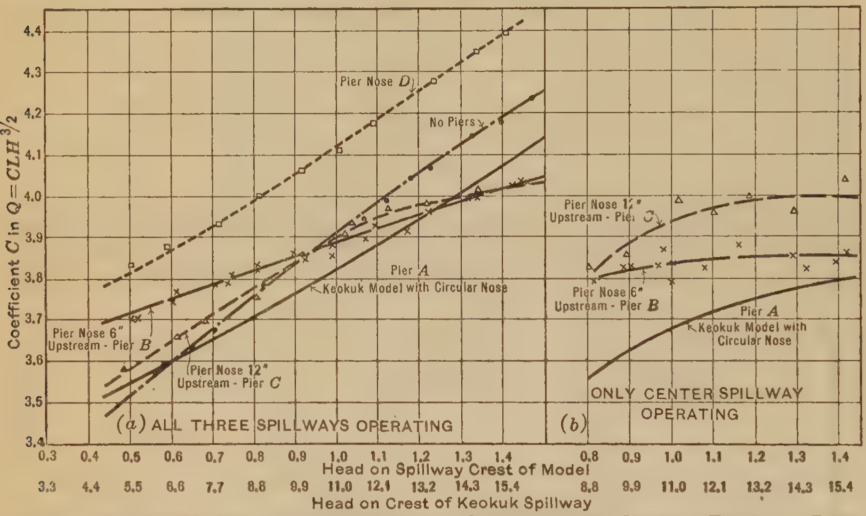

FIG. 25.—DISCHARGE COEFFICIENTS FOR KEOKUK SPILLWAY WITH VARIOUS FORMS OF PIERS.

The Keokuk spillways are 30 ft. wide and the piers, 6 ft. thick. The relative amount of obstruction caused by the different forms of piers may be expressed more clearly by tabulating the discharge over a dam of a certain gross length made up of these piers and spillways with that which would pass over a dam of the same gross length with no such obstructions (see Table 9).

TABLE 9.—DISCHARGE OVER DAM WITH PIERS EXPRESSED IN PERCENTAGE OF DISCHARGE OVER DAM OF SAME LENGTH WITH NO PIERS. ALL SPILLWAYS IN OPERATION.

(Piers and spillways similar to Keokuk Dam assumed. Open spillway equals 0.83% of the gross length of the dam.)

Type of pier.	Head = 5 ft.	Discharge in Percentage of That Through Unobstructed Spillway.	
		Head = 11 ft.*	Head = 15 ft.
No piers	100	100	100
Keokuk Piers:			
Pier *A*	84	81	81
Pier *B*	88	83	80
Pier *C*	83	83	80
Pier *D*	91	88	87

* Normal head at Keokuk.

The small differences between the discharge over the spillways with various pier forms when all spillways were in operation were greatly magnified when a single spillway was operating alone. Tests with the center spillway of the model operating while the other two were closed by gates were performed with Piers *B* and *C*, and had previously been made upon the Keokuk Pier *A*. The results are shown in Fig. 25 (*b*). The curves in this diagram show a decided increase in the discharge accomplished by projecting the pier ahead of the dam. Pier *C* allows about 7% more water to pass through a single spillway than passed over the spillway when operating as a model of the Keokuk Dam. At the lower heads, Pier *B* gave discharge coefficients similar to Pier *C* which had twice the amount of projection ahead of the dam, but, at the high heads, Pier *B* gave results closer to those of the Keokuk model, indicating a positive advantage in the further extension of the pier into the pond.

When the piers extended into the pond, the water surface of the nappe as it passed over the crest was not greatly depressed at the spillway sides. However, Fig. 26 which shows the surface of the nappe at three different sections (only center spillway open) indicates that although the nappe is comparatively flat above the crest at the gate section, 6 in. ahead of this at the face of the dam, the surface is appreciably depressed along the pier walls. The comparison between the nappe surface at the metering section with that obtained on the Keokuk model is interesting. Except at the spillway sides, the nappe produced by Pier *B* is not so thick, but since the total discharge is greater, interior pressures must be less and current velocities more.

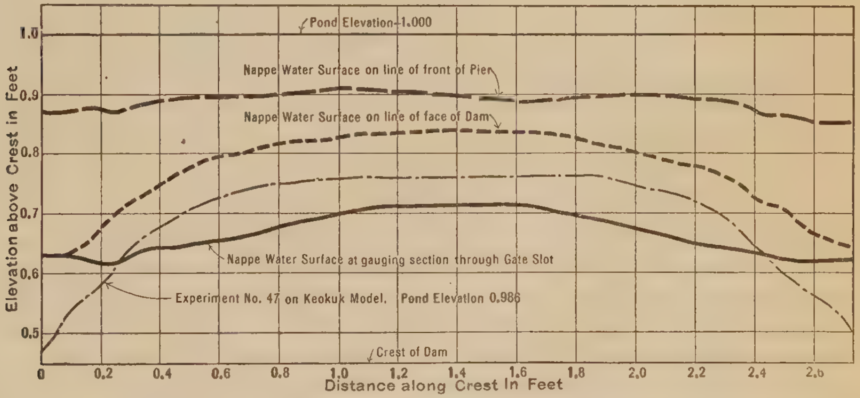

FIG. 26.—CENTER SPILLWAY WITH PIER *B*, COMPARED WITH NAPPE OF KEOKUK MODEL.

ACKNOWLEDGMENTS

These experiments were financed by the Mississippi River Power Company and the Hydraulic Laboratory of the State University of Iowa.

S. M. Woodward, M. Am. Soc. C. E., of the Department of Mechanics and Hydraulics, State University of Iowa, made valuable suggestions in the preparation of this paper and with J. B. Spiegel, M. Am. Soc. C. E., District Engineer, U. S. Geological Survey, visited Keokuk and inspected the

work during the progress of the experiments on the spillways. Mr. Paul Mercer, Assistant Engineer, Mississippi River Power Company, and George E. Shafer, Jun. Am. Soc. C. E. (now Assoc. M. Am. Soc. C. E.), Research Assistant in Hydraulics at the State University of Iowa, assisted in making the measurements at Keokuk. The current meters were rated for these tests free of charge by the engineers of the U. S. Bureau of Standards. The following graduate students in Hydraulic Engineering at the State University of Iowa served as observers in performing the model experiments and assisted in the preparation of this paper: T. L. Herrick and J. W. Howe, Assoc. Members, Am. Soc. C. E., W. E. Smith, J. W. Hummer, Glen N. Cox, G. H. Hickox, and J. S. Meyers, Juniors, Am. Soc. C. E., and Messrs. Arnold Neshein, Ned Ashton, H. C. Brockman, and J. C. Ducommun.

DISCUSSION

B. F. Jakobsen,* M. Am. Soc. C. E. (by letter).—These tests are of particular interest, because they furnish information regarding the discharge coefficient for heads up to 11 ft. for the Keokuk spillways. A head of 1 ft. on the model corresponds to 11 ft. on the prototype, and the agreement between the model and the actual structure is quite close (Fig. 20). It would be interesting to have some probable errors calculated for a number of these tests and to determine the coefficient of correlation between the tests on the model and those on the actual structure.

The tests suggests that the discharge coefficient of the Francis formula, $Q = C\,L\,H^{1.5}$, is not a function of the head, but of some ratio of, or relation between, the length and the head. It is possible that the authors may be able to derive some such relation from the tests already made. If not, additional tests could be made, for example, by introducing a portable pier at Keokuk and also in connection with the model, so as to be able to vary the ratio between length and head. If the authors could establish some such relation, it would be of considerable practical value, since the discharge coefficient, C, varies from 3 to 4.

With reference to Figs. 12 and 13, it would be instructive to have the value of C plotted for a number of vertical sections, beginning as close as possible to the piers.

It appears that the authors use the term, "actual velocity", when they mean "maximum velocity". For example, they state: "The contours in Figs. 12 and 13 * * * show the actual velocity of the water which was measured throughout the nappe."

From what follows in the text, the writer infers that the reference is to the maximum velocity, which, however, was not measured, but calculated from measurements. As the writer understands it, the horizontal velocity is as actual as the velocity in any other direction. The influence of the shape of the piers, as noted by the authors, should also be carefully studied.

J. B. Spiegel,† M. Am. Soc. C. E. (by letter).—The authors have made a valuable contribution to engineering knowledge by their presentation of the results of the experiments and tests of weir discharge performed at the Keokuk Dam. After reading the paper one can readily understand the reasons that delayed its appearance until five years after the experiments were finished. Those who are constantly studying the behavior of surface waters and who realize how applicable this information is to numerous problems that require solution will be pleased to learn that they may now select coefficients for high-head weir discharges with greater confidence. Uncertainty has always lurked in the minds of engineers concerning the influence of the opening of neighboring gates on individual weir discharges.

Fig. 14 reveals a relationship that is indeed interesting, but leads one to wonder just where the central-weir velocity curves would plot for the condition of nine, eleven, thirteen, etc., gate openings. Apparently, the greater the number of openings the closer the results approach the curve of velocities corresponding to the head below pond level. It is appreciated that a greater number of openings at the time these observations were taken probably would have seriously affected the efficient operation of the power plant and would have been wasteful of water.

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† Dist. Engr., U. S. Geological Survey, Topeka, Kans.

Since practically all velocity measurements made by the U. S. Geological Survey are obtained with Price current meters, a statement of results as compared with the registered or adjusted values of the Ott meter would be of interest to its engineers. Velocities of more than 12 ft. per sec. are rarely exceeded in ordinary stream-gauging work and no appreciable wear on the meters is noted except when they are used in waters heavily charged with silt.

The Keokuk Dam is situated at a section that carries the discharge from about 119 000 sq. miles of drainage area. The component areas have not been studied comprehensively or continuously. Therefore, no reliable estimate of flow can be obtained, except by use of the Keokuk records and these experiments assure the accuracy of these records for at least the 16-year period of operation of the plant. Gauge-height observations and some discharge measurements on the Mississippi River at this location prior to that period and extending back to the late Seventies afford a record of fifty years or more, which is a valuable contribution to the study of the perplexing Mississippi River flood-control problem.

Bulletin No. 14, "The Control of Floods by Reservoirs", issued by the State Engineer of California and the conclusions of the Vermont Flood Commission,* indicate that engineering thought is directed toward the study of the feasibility of co-ordinating water-power development and flood control. The perfection of discharge formulas for application in such projects, on which weirs often play an important part, is timely and welcome.

DANA M. WOOD,† M. AM. SOC. C. E. (by letter).—This paper brings forcibly to consideration the behavior of current meters when subjected to angular flow conditions.

The ideal meter would be one which, when held rigidly in a horizontal plane and at right angles to the measuring section, would correctly measure that component of the velocity normal to the section. It is to be noted that in the Keokuk experiments, the meter was allowed to swing in a horizontal plane to meet the direction of the average current striking it, the angularity being determined by an indicating device. The meter was not permitted to swing in a vertical plane, except in two special tests. Therefore, the effect of variations in vertical angularity and of minor departures from the average or predominating horizontal angularity, at any point of measurement, was taken account of only in the standard behavior of the particular meter used under the composite conditions existing, with a standard correction obtained from Fig. 9.

For some time it has been known that certain of the standard makes of current meters over-register, and that others under-register, the true normal velocity when held rigidly in a normal horizontal position and when currents are oblique to the meter in either a horizontal or vertical plane, or a combination of each. To what extent in the Keokuk experiments correction for horizontal angularity only accounts for all possible over or under-registry, is not made clear in the paper. It hardly seems possible that Fig. 9 is applicable to all stations, especially those near the piers.

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* *Engineering News-Record*, p. 72, Vol. 102, No. 2.

† Hydr. Engr., Stone & Webster Eng. Corporation, Boston, Mass.

Furthermore, there is usually not only a difference in the error of registration resulting from horizontal and vertical angularity of currents, but also a difference from currents hitting the meter from the left and from those hitting it from the right. Variations of methods of suspension, and size and shapes of weights, if they are used, affect meter ratings and registration, as has been well proved by the Water Resources Branch of the U. S. Geological Survey. It is open to question whether one standard laboratory correction curve adequately covers variations actually occurring during the measurements.

The matter of whether or not full consideration has been given to all these questions, or whether their effect has been determined to be negligible, in view of other errors of measurement possible, is not made clear in the paper.

It is very important to select the proper type and make of current meter when used for experiments as described. While the Ott type of meter may have been best adapted to the Keokuk conditions, such general statements in favor of its selection as "it measures the normal component of angular currents more accurately than other types", might be amplified for clearness. If the meter is allowed to swing in either a horizontal or vertical plane, and its axis assumes a position parallel to the average stream line, the statement would refer to a plane "normal to the meter" rather than "normal to the measuring section", although the latter is the desired velocity. Accurate results can be obtained only when the direction which the axis of the meter assumes can be reliably ascertained. There may well be a set of conditions in which another type of meter might be better suited for determining the velocity normal to the measuring section even if the Ott type proved best at Keokuk.

The authors state that "with the 3-blade propeller it [the Mensing-Ott meter] had characteristics in measuring angular currents which were superior to the others". In view of the paucity of experimental data on the behavior of current meters under conditions of angular velocity, would it not be well if the authors developed the basis for this statement more fully?

With the lower actual velocity and depth conditions in the model tests, is it safe to assume that, because of the apparent verification of Pitot tube checks, the limit of accuracy of the field tests was within 2 per cent? In view of the care taken in all the work and the study given to the many complicated problems, the conclusion may be correct, but the writer would like to see some discussion from others who have conducted laboratory and field tests on these questions.

S. P. WING,* Assoc. M. Am. Soc. C. E. (by letter).—Practical hydraulics, being largely an empirical science and one in which the required coefficients have been determined for the most part from tests on structures of a size far smaller than those used in practice, has been handicapped for the lack of adequate design data. At the time this paper was published, the writer was endeavoring to fix the coefficient of discharge for a large overflow dam. By making use of the data presented he was enabled to make a direct saving in the size of his proposed flood-gates. The work of the authors is of particular value, in that experiments were made on both a model and on the full-sized structure.

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* Engr., Constr. Dept., British Columbia Elec. Ry., Vancouver, B. C., Canada.

Although the theory of models is well established, certain factors cannot be adequately reduced to scale and tests of the full-scale structure remain the final criterion. The high degree of correlation obtained by the authors, between the results of the model and those of the full-scale structure, means that future tests on similar structures can be confined to those made on the cheaper models.

The tests on the model at high heads with the coefficients, as shown in Fig. 19, are of special interest. These reach the value of 4.15 and the curve gives no indication of flattening. The upper limit should be reached when the depth of overflow arrives at a point at which the nappe jumps clear of the face; it would be interesting to know what this limit is. It would also be of interest to know at what point a vacuum begins to form below the nappe.

The economic design of the crest of an overflow dam has two main controlling factors: First, the design should be such that the coefficient of discharge will be as large as possible; and, second, such that no vacuum will take place under the overfalling sheet. With present information as to the shape of the lower nappe, based on Bazin's experiments and as applied in Creager and Justin's Hydro-Electric Handbook, if vacuum is to be prevented, the relation of the depth of overflow with respect to the curve of the dam crest must be such that the coefficient of discharge is limited to about 3.9 or 4.0 for an unobstructed crest. It would be of interest to know whether this limit exists in fact, as the profile and the cost of an overflow dam are controlled largely by this consideration.

In applying the results of these tests to other structures, it becomes necessary to find the scale at which the proposed structure most closely resembles that on which the tests were made, and then to make use of this scale in determining the proper discharge coefficient. This fact occasionally has been overlooked, data being published as to the value of the coefficient for certain profiles or for certain depths of flow without giving the full dimensions of the structures and the depths of flow at which the tests were made. In fitting a proposed profile to a test profile to find the scale relationship, probably most weight should be given to the fit of the up-stream face. Small differences at that point have a large effect on the coefficient of discharge as was shown in the model experiments* made at Cornell University. Additional experiments are needed as to the effect of varying the down-stream profile.

The authors have shown that results with a high degree of accuracy can be obtained from relatively inexpensive hydraulic models. The writer hopes that they and others will make additional tests to obtain coefficients for the lack of which, hydro-electric designers now must use too much guesswork. A foot of head frequently is worth $100 000, and the economic design of a flow line deserves better data than are now available.

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* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1199.

E. A. Rudulph,* Assoc. M. Am. Soc. C. E. (by letter).—This paper is particularly timely in view of the growing interest now being manifested by hydraulic engineers in the use of models for the solution of practical hydraulic problems.

The great difficulty and high cost of making accurate measurements of the large quantities of water and high velocities usually encountered in hydraulic structures, have resulted in a scarcity of conclusive information to demonstrate the agreement between model and full-scale measurements. In consequence, engineers have hesitated to adopt the results of experiments on models as a reliable measure of full-scale hydraulic conditions.

The close agreement between model and full-scale measurements obtained through the accurate and painstaking work done at Keokuk should go far toward establishing confidence in results obtained from model tests. Moreover, in view of the large-scale model used by the authors, and the greater possibility of error in the experiments on the full-sized structure, it is possible that the most reliable discharge coefficients of the Keokuk spillway are those determined by the model test.

An essential requirement in any hydraulic model test is that true similarity between model and full-scale conditions be attained. The same reasons which cause engineers to hesitate in accepting model-test results frequently cause them to select, when such experiments are attempted, the largest scale for the model that laboratory facilities will permit. In so doing the true hydraulic similarity between the model and its full-scale prototype is often overlooked. The following example from the work on the Keokuk model illustrates this point.

While the head on the crest of the Keokuk spillway was measured at points where little or no velocity normal to the dam existed and, therefore, required no correction for the velocity of approach, the authors do not indicate that the results obtained from the model were corrected for velocity of approach in the test flume. With all three gates open this velocity would have corresponded to about 1.2% of the normal head of 1 ft. on the crest of the model. Applying this correction to the coefficient obtained for three gate openings, it is reduced to approximately 3.75, a value somewhat more reasonable in the light of Tests 4 and 5 on the full-sized structure. With only one gate on the model open, the velocity of approach becomes so small as to be negligible, and no correction is required. If a model of smaller scale had been used, a greater number of gates could have been accommodated in the 10-ft. width of the test flume and the approach conditions to the spillway could have been made to simulate more truly those of the full-sized structure. The head measurements could then have been made in a location similar to those for the actual spillway, and no velocity of approach correction would have been involved.

Greater precision of measurements will naturally follow from the use of large-scale models, assuming adequate laboratory facilities for measuring accurately the larger quantities of water used. Moreover, the comprehensive study of flow conditions within the overfalling sheet made by the authors would require a large-size model. Ordinarily, however, entirely satisfactory quantitative determinations can be obtained from models of much smaller scale than were used for the Keokuk tests.

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* Engr., Stone & Webster Eng. Corporation, Eldon, Mo.

Experiments in European laboratories have demonstrated that as long as the velocity remains sufficiently high to insure a turbulent or rolling condition of flow, the laws of hydraulic similarity continue to apply. Any difference in results obtained by models of different scales, therefore, will be due to the relative frictional resistance to flow. The discharge of a spillway is affected by frictional resistance of the surface of the structure only in the region of low velocity up stream from, and a short distance over, the crest. Below this point the surface of the overfalling sheet falls below the critical depth, and frictional resistance no longer has any effect on the discharge. For this reason the effect of the relative roughness of spillway models is negligible, a fact demonstrated by the authors by comparing the discharge coefficients obtained from sanded and smooth-surfaced models. The use of a small model, therefore, will affect the accuracy of the results only in so far as it affects the accuracy of the head and discharge measurements. Aside from securing true hydraulic similarity the use of small-scale models has the added advantage of lower cost and increased facility of testing.

The reliability of the model tests having been established by comparison with the test on a full-sized structure, the importance of making model experiments in the design of hydraulic structures is strikingly demonstrated by the experiments performed by the authors, which show the possibility of materially increasing the discharge capacity of the spillway by the adoption of properly shaped piers.

C. H. PIERCE,* M. AM. SOC. C. E. (by letter).—Ratings of hydraulic structures by means of laboratory measurements on small scale models form one of the most interesting and fascinating subjects in the field of hydraulic research. Experiments and laboratory tests of models have become fairly common in occurrence, but rarely is the opportunity available for comparison of laboratory experiments and tests with the results of measurements made on the full-sized structure under operating conditions. Therefore, the experiments described in this paper should be of especial value.

Some of the earlier tests on models which have come to the writer's attention indicated that the requirements of mechanical and hydraulic similitude were not always fully complied with, and, consequently, the results of the tests were not conclusive. It is possible also that all the laws of hydraulic similitude may not as yet be established or understood. For example, it is only recently that the foremost experimenters in this subject have fully comprehended the time relation for motion picture exposures required for an undistorted time scale from the model.

In 1918, Benjamin F. Groat, M. Am. Soc. C. E., gave a statement† of the "Theory of Hydraulic Models", and showed the derivation of some of the fundamental relations which exist between the full-sized object and the model. More recent expositions of these relations have been made by Professor Hubert Engels and others.*

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* Care, Charles T. Main, Inc., Boston, Mass.

† "Ice Diversion, Hydraulic Models, and Hydraulic Similarity," *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), pp. 1108–1190.

A condensed restatement of the known laws of hydraulic similitude may be permissible. For a model built to a scale relation of 1 : *k*, the coefficients to be applied to the model features in order to transfer them to the full-sized structure are as follows:†

Coefficient.	Kind of Quantities to Be Transferred.
$k^{-0.5}$	Number of revolutions of water-wheels (pumps, turbines); number of motion picture exposures for undistorted time scale.
k^{0}	Constants; ratios (obstruction ratios, flow ratios); relative slopes; unit weights; accelerations.
$k^{0.5}$	Time; velocities.
$k^{1.0}$	Lengths; widths; heights; absolute slopes; velocity heads; friction heads; stresses; strains; wind pressure; towing resistance per unit of area; temperature changes through frictional resistance.
$k^{1.5}$	Discharge volume per unit of time and of width.
k^{2}	Areas (cross-sections).
$k^{2.5}$	Discharge quantities per unit of time for the entire stream.
k^{3}	Cubic contents; water volumes; mass forces (weights, impact, towing forces, wind pressures, reactions).
$k^{3.5}$	Work done per unit of time; magnitudes of movements (impulses).
k^{4}	Movements; work; energies (potential and kinetic energies).

The velocity with which the water approaches the crest is one of the major factors which determine the discharge equation of the spillway, and it is essential that the approach channel of the model shall be constructed so that the relation of velocity heads will conform with the law of similitude. Another important consideration is that the relative slopes of the water surfaces shall be in conformity. The fulfillment of this latter requirement of itself will assure the proper degree of smoothness of the model.

It would be manifestly impossible to reproduce, in an hydraulic laboratory of an engineering school, all the diverse conditions pertaining to the 119 spillway sections which extend across the Mississippi River from the Illinois shore to the power house at Keokuk, Iowa. Certain typical conditions can be evaluated, however, and general conclusions drawn from them. This appears to have been the method of attacking the problem at Keokuk, with respect to the current-meter measurements and the model tests.

Eight current-meter measurements were made of discharge over one spillway (Table 1). These measurements may be arranged in five groups as follows:

(*a*) No adjacent gates open (Measurements Nos. 1, 2, and 8).
(*b*) One adjacent gate open (Measurement No. 3).
(*c*) Two adjacent gates open (Measurements Nos. 4 and 5).
(*d*) Four adjacent gates open (Measurement No. 6).
(*e*) Six adjacent gates open (Measurement No. 7).

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* See "Hydraulic Laboratory Practice," edited by John R. Freeman, Past-President, Am. Soc. C. E.; and pub. by the Am. Soc. of Mech. Engrs., January, 1929.

† Arranged from notes of lectures given by Prof. Dr.-Ing. Theodor Rehbock, at Mass. Inst. Tech., March, 1929.

Because of the small range in head between the highest and lowest measurements, (0.37 ft. between the head of 10.99 ft. for Measurement No. 7, and 10.62 ft. for Measurement No. 5), it is necessary to plot them at an extended scale in order to analyze the results graphically. This has been done in Fig. 27, in which logarithms of head have been plotted against logarithms of discharge, using ordinary cross-section paper. The results of this plotting are fairly consistent. Measurement No. 7, which probably had the greatest velocity head on account of the large number of open gates, plots highest on the discharge curve. Group (*a*), with no adjacent gates open, is consistent with respect to location of the group, although the individual measurements differ somewhat among themselves. Measurement No. 3, with one adjacent gate open, shows a slightly larger discharge coefficient than the average of Group (*a*), which is as might be expected. The straight line defined by Group (*c*) shows a still larger coefficient, due to the greater velocity head. At first glance, Measurement No. 6, with four gates open, does not appear to be consistent, until it is noted that the four gates were all on one side of the section, and, consequently, the velocity head at the measuring section was probably about the same as for the condition of one adjacent gate open, as in the case of Measurement No. 3. Measurement No. 7, with six adjacent gates open, and with three gates on each side of the measuring section, shows the highest coefficient of all, about 2.839, with 1.628 as the exponent of the head.

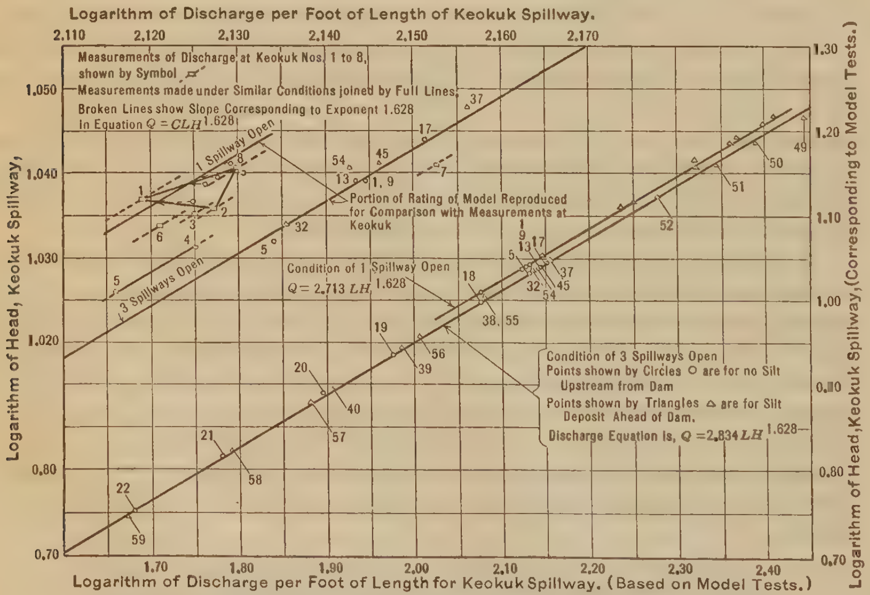

FIG. 27.—RESULTS OF LABORATORY TESTS ON MODEL OF KEOKUK DAM COMPUTED FOR DISCHARGE PER FOOT OF LENGTH OF KEOKUK SPILLWAY. RATIO, $n = 11$.

In studying the rating of the model, it will be seen that for the condition of three spillways open, all the measurements made under that condition plot very close to a line the equation of which is $Q = 2.834\ L\ H^{1.628}$. The condition of silt ahead of the dam appears to have given results not appreciably different from the condition of no silt deposit. For the condition of one spillway open, the measurements define a line having the equation, $Q = 2.713\ L\ H^{1.628}$.

It should be noted that the small group of measurements with one spillway open, which are reproduced at the enlarged scale between the heads having logarithms of 1.035 and 1.041 plot somewhat larger than the average curve for that condition. It is believed, however, that the average curve is more dependable than individual measurements or any one group of observations.

For those who desire to use $H^{\frac{3}{2}}$ and a coefficient varying with the head instead of the exponential equation as derived from a straight-line log-diagram, the writer would suggest that it is easy to recast the equations mentioned into the form, $Q = C_{\text{var.}}\ L\ H^{\frac{3}{2}}$. The writer believes that logarithmic plotting affords the simplest and most reliable means of analyzing and evaluating many types of experimental data, especially those obtained from hydraulic experiments.

It is gratifying to note the considerable number of engineering schools and engineering experiment stations which now have equipment for conducting hydraulic experiments on a large scale. These laboratories, with the support and encouragement of engineers who are interested in hydraulic research, can be developed so as to provide ample facilities for the study of any hydraulic problem.

The authors have given the results of a series of very interesting and useful tests made at the laboratory of the University of Iowa. It is probable that there are numerous experiments made at other hydraulic laboratories which would be of value to the Engineering Profession if those in charge of the laboratories could find time to arrange the data for publication.

DR. ING. LUDWIG A. OTT[*] (by letter).—This paper is of interest to the writer because it is the first that describes a case in which discharge measurements with current meters have been made under such extraordinary conditions and with such satisfactory results.

The particular difficulties in the measurements at the Keokuk Dam were caused by the presence of very high water velocities combined with great depths and oblique currents.

In high water velocities and at great depths the mechanical stress on a current meter rod of the usual circular cross-section is far greater than one is inclined to believe, if one has not made this problem a special study. Besides, even in uniformly smooth currents, high lateral stress may frequently occur, that may cause intense vibrations of the rod.[†] Rods of stream-line shaped cross-section are by far the most serviceable. Under equal conditions of water velocity and width of section, the pressure on such a rod is only about one-fifth that on a rod of circular section.

In regard to the current meters themselves, the Ott meters, used in the researches, will withstand the greatest velocities without danger of injury.

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[*] Math. Mech. Institut, Kempten, Bavaria, Germany.

[†] See F. Eisner, "Widerstandsmessungen in umströmten Zylindern", *Mitteilungen* der preuss. Versuchsanstalt für Wasserbau und Schiffbau, No. 4, Berlin, 1929.

The most noteworthy examples in this respect are perhaps the measurements reported by F. Kuntschen.* The meter used in these experiments with entire success at a water velocity of 42 ft. per sec. (corresponding to more than 6 000 rev. per min.) was of the same type as the instrument used at the University of Iowa for velocity measurements in its experiments on the weir model.

In the case of extended measurements under very high stress, however, the particular Ott meters used at the Keokuk Dam are scarcely the most serviceable, because their axle bearings are not devised for such an extraordinary stress for long hours. It would be better to use an Ott meter of a type provided with steel ball-bearings running in an oil bath. This construction permits using the meter with confidence in extremely high velocities without affecting in the least its sensitiveness for a current as slow as about 0.15 ft. per sec.

A third difficulty in making measurements at the Keokuk Dam (which was also overcome satisfactorily) was the existing oblique currents.

The execution of measurements in oblique currents is the more simple and exact, the more closely the behavior of the screw follows the cosine law. Experience has taught that in this respect conical screws with two blades give less accurate results than cylindrical screws with three blades attached to spokes, as has also been proved by the U. S. Bureau of Standards ratings of the meters used. The narrowness of space alone was responsible for the use of the 2-blade, instead of the 3-blade screw, 7 in. in diameter. In connection with the meter equipped with ball-bearings in an oil bath, cylindrical screws, 4 in. in diameter, are furnished which show a particularly small deviation from the cosine law. At an angle of 20° between the direction of flow and the axle of the meter, the correction to be added to the determined velocity (see Fig. 6) is only 2 per cent.

In making the statement that the measurements at the Keokuk Dam might have been carried out more advantageously by the use of a rod with a streamline shaped section and an Ott meter of different type with a 3-blade cylindrical screw, 4 in. in diameter, the writer, of course, intends no reflection on the accuracy of the results obtained by the means used. On the contrary, he considers them to be highly satisfactory.

The comparative measurements at the Keokuk Dam and on a model of the spillways are similar to measurements carried out by the Research Institute for Hydraulic and Hydro-Electric Structures (Forschungsinstitut für Wasserbau und Wasserkraft), in Munich, Germany, at the spillways of the Mittlere Isar River and on a model one-fiftieth of the full size.† In this case the comparison of the results obtained on models and at the actual structure also produced a very satisfactory confirmation of the model rule.

At the writer's request, Dr. Ing. O. Kirschmer, Chief Engineer of the Research Institute for Hydraulic and Hydro-Electric Structures, has compared the results of his measurements with those at the Keokuk Dam with satisfactory results, as follows:

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* "Le moulinet hydrométrique et la mésure de courants très rapides," Communications, Suisse Service Fédéral des Eaux, No. 18, Berne, 1926.

† A report on these experiments is contained in "Hydraulic Laboratory Practice" (pub. by the Am. Soc. Mech. Engrs.), pp. 442-453, under the heading "Experiments to Determine the Discharge Coefficients of Certain Weirs with Round Crests".

During its experiments the Research Institute for Hydraulic and Hydro-Electric Structures has effected many changes in the forms of the weirs. There was little difficulty in comparing the results obtained with those at the Keokuk Dam. After reducing the weir coefficient determined by the Research Institute to existing conditions at the dam, the results obtained in both studies show a very satisfactory agreement. All the results determined on the model at the University of Iowa lay well within the range of "scattering" values observed in the experiments of the Research Institute (see shaded area in Fig. 28).

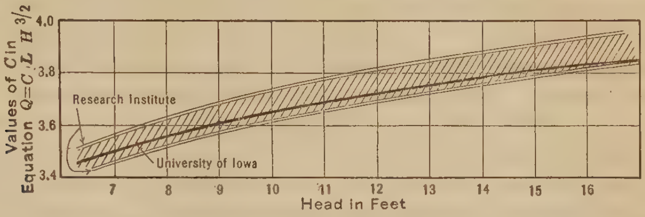

FIG. 28.—COMPARISON OF THE MODEL EXPERIMENTS OF THE UNIVERSITY OF IOWA AND THE RESEARCH INSTITUTE FOR HYDRAULIC AND WATER POWER ENGINEERING, MUNICH, GERMANY.

After very extended studies the Institute has reached the conclusion that on weirs certain variations in the flow condition will arise (instability of the current), and that, owing to this circumstance, various coefficients may be determined under fixed conditions at one and the same weir. For this reason, Dr. Kirschmer has not drawn in Fig. 28 a reduced or mean curve of weir coefficients, but has shown only the upper and lower limit between which the coefficients vary, owing to the unstable condition of the current. If one draws a mean curve within these limits, there remains a negligible difference between this curve and the one obtained by the authors which, perhaps, may be accounted for by the different methods of measuring (volumetric measurements at the Institute and Bazin weir measurements at the University of Iowa). Incidentally, in Germany the Bazin weir formula is now scarcely used for the computation of discharges.

The experiments at the Keokuk Dam can be compared further with those of the Research Institute at the spillway of the Mittlere Isar River with regard to the influence of obstructions, such as piers. In this case there is likewise an agreement between the authors' and Dr. Kirschmer's experiments. This influence on the weir coefficients appears to be somewhat less in the experiments of the Institute than in those at the Keokuk Dam, but it must be remembered that at the Mittlere Isar the thickness of the piers is also less in proportion to the width of the spillways at the Keokuk Dam.

The agreement between the model experiments and the measurement at the actual structure obtained on both parts may also be considered an indirect proof of the reliability of the current-meter method of measuring discharge. By virtue of its previous experience with regard to current-meter measurements the Research Institute is convinced that, with meters of suitable construction, great accuracy of results can be obtained even in very turbulent currents. In order to gain conclusive evidence as to the absolute accuracy of current-meter measurements, the Institute intends to make extended experiments by comparison with the volumetric method. For this purpose the Research Institute has provided for the erection at its laboratory near the Walchensee a measuring tank with a capacity of 53 000 cu. ft., which will soon be put to use.*

Investigations concerning the accuracy of the various methods of discharge measurements form an essential item in the program of the Research Institute.

B. F. Groat,† M. Am. Soc. C. E. (by letter).—It is doubtful whether most engineers can appreciate the difficulties of real pioneer investigation. Most of the great works of engineering were erected from plans based on well-tried methods—"accepted practice". This is eminently proper, because lives and dollars should not be subjected to unnecessary risk by structures built on sands of misapplied theory. Nevertheless, spillway discharge has been for years a subject of more, rather than less, speculation as to just what the coefficient should be, especially for high heads. In this paper the authors have joined hands in a real pioneer investigation, the difficulties of which will be fully realized by all engineers who have had opportunity to conduct such researches.

The conditions surrounding experimental work are so complicated and interwoven in relationships that it is often necessary to expend a great amount of time, labor, and experimental mechanism, before any useful result can be obtained. Sometimes, an apparently simple device will be used for years before its unreliability is fully and generally suspected. This was the case with current meters when used for precise water measurements. One of the first, if not the first, to notice the effect of turbulent flow in causing a change of current-meter ratings was the late Charles H. Miller, M. Am. Soc. C. E.‡

Notwithstanding the many criticisms made by later observers, the current meter is to-day frequently used by engineers without regard to Miller's words of caution. It is, therefore, a source of satisfaction to know that the careful experimenters at Keokuk have applied corrections for errors due to angular disposition of the meters. The writer has also made similar efforts to correct the errors of current meters.

In one extensive series of experiments the discharge of large turbines at Massena, N. Y., was measured simultaneously by current meters and chemicals. These comparisons were from the earlier and least accurate of the chemical tests, but, even in these, the results show a high degree of precision in both methods of measurement.§ Omitting three tests in which there were known faults in the laboratory, or apparatus, the results were as given in Table 10.

The current meters used in these tests consisted of three different types, namely, one small Price meter, two Haskell meters, and two Ott meters. They operated on a movable frame, so that, by shifting once at each elevation, the five meters gave records for ten different points of observation in the cross-section. As there were ten elevations at which the meters operated, there were, in all, 100 velocity readings for one complete sweep of the frame, which corresponds with one test.*

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* For a description of the Research Institute of Hydraulic and Hydro-Electric Structures, see "The Research Institute for Hydraulic and Hydro-Electric Structures", by B. R. Van Lear. *Mechanical Engineering,* Vol. 50, No. 8, August, 1928, pp. 607-611.

† Cons. Engr., Brookline, Mass.

‡ *Engineering News,* January 24, 1885, p. 58; also, "The Discharge of the Mississippi River," by William Starling, M. Am. Soc. C. E., *Transactions,* Vol. XXXIV (1895), p. 367.

§ "Chemi-Hydrometry and the Precise Testing of Hydro-Electric Generators," *Transactions,* Am. Soc. C. E., Vol. LXXX (1916), p. 1270, Table 92.

No such satisfactory agreement could possibly have been attained in these tests had the ratings of the meters not been carefully corrected for errors. This will be appreciated when it is stated that some of the meters (depending on their type and their position in the raceway) diverged from their true ratings by amounts varying from + 10 to — 25 per cent.

TABLE 10.—COMPARISONS OF DISCHARGE MEASUREMENTS.

Test No.	DISCHARGE DETERMINED BY:	
	Current meter, in cubic feet per second.	Chemical method, in cubic feet per second.
G	1 663	1 662
H	1 662	1 662
I	1 662	1 666
K	1 618	1 683
L	1 648	1 647
N	1 785	1 777
O	1 783	1 768
S	1 676	1 640
Total	13 497	13 505

The meters were held rigidly with axes pointing up stream, parallel to the axis of the raceway in all cases. The writer is sure that a meter which is allowed to swivel about its support is not as reliable as one fixed in its position, ratings in both cases being corrected for turbulence. Moreover, since there is no meter that gives a correct resolved component of motion, engineers should always use at least two types of meter simultaneously. One of these should be a screw type and the other a cup, because the former is retarded and the latter accelerated. This is not to be considered a criticism of the method of correcting ratings adopted by the authors, however, because the case in hand is rendered far simpler than in one in which there is great turbulence, as shown by the statement that,

> "The water being accelerated as it passed over the crest, flowed with remarkable uniformity in smooth stream lines and was free from the characteristic pulsations in velocity and annoying vortices which so often accompany the phenomenon of flow in other channels."

However this may be, the writer believes he is perfectly correct in the following propositions which ought to be considered as theorems of hydraulics:

1. (1) Meters should be held rigidly.
2. (2) A "rudder", or "tail", is a monstrous absurdity.
3. (3) A "screw" type of meter can be designed to give the resolved component.

These statements are not directed toward manufacturers, but to engineers as a whole who take altogether too long a time to make the most of past experience. It is about 250 years since, in a single paragraph, Newton enunciated the principles of similarity and the theory of models, to which no man has been able to add one jot of additional theory; yet it is only in the last very few years that engineers have made any use of models as a reliable means for determining a design, especially in hydraulics. Charles Warren Hunt, M. Am. Soc. C. E., has stated* the fact very tersely as follows: "* * * a model is the only means for reproducing all the complexities of flow caused by natural channel conditions".

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* "Chemi-Hydrometry and the Precise Testing of Hydro-Electric Generators," *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1270, Table 92.

It is important to know how small the model may be made without loss of value as a reliable criterion. This is a question of special importance relative to experiments in which a long waterway is involved. In such cases the effects of friction are of much greater account than they are for weir, or spillway discharge, in which there is very little chance for frictional resistances, owing to the short length of the waterway involved.

As a matter of fact, the coefficient for frictionless weir discharge could not differ greatly from the coefficient determined by experiment, since the effect of friction is practically nothing for a weir. When the statement is made that the coefficient for frictionless flow would be about the same as for an experiment, it must be remembered that the equation, $Q = C\,L\,h^{\frac{3}{2}}$, is not a theoretical formula, but an empirical one. Therefore, the coefficient, C, for frictionless flow—instead of being unity—would express the numerical value of the function of H which would convert the empirical formula into the correct theoretical formula. That function, if correct in form, would undoubtedly eliminate H in terms of a true "head on the weir".

Before considering a weir and its model it will be desirable to acquire an understanding of the true nature of hydraulic friction. In the first place, there cannot possibly be any ordinary "sliding friction" of a fluid along the surface of a solid where there is contact. This theorem has been proved by many experimenters, notably first by Poiseuille in his celebrated experiments on the flow of fluids through capillary tubes. In cases of turbulent flow the velocity a short distance from the surface may be relatively high; but the presence of the "film" is certain, although its thickness may be very slight. The thickness of the film depends on the neighboring velocity of the fluid, the fluid itself, and the character and condition of the surface.

It is this thinning of the film (due to motion near-by) of steam, water, or air, along the side of a radiator which increases the heat conductivity of the radiator enormously. As a matter of established fact, the great resistance to heat transmission through good conductors is due principally to the thermal resistances of the fluid films on the two sides of the conductor, rather than to the resistances of the conductors themselves. For example, water boils only a little more quickly in a thin metal pot than in one of the same material several times thicker, or than in one made of thin asbestos, say. Asbestos is a poor conductor as regards the flow of heat through its material, but, for a thin sheet of asbestos, the resistance of its material is much less than the resistances of the fluid films on the two sides, even if the water be in turbulent motion. No correct estimate of heat conduction through walls from one medium to another can be made unless the films of the media on the walls of the conductor be taken into account. In many cases, the resistances of the conducting materials themselves can actually be omitted without serious error.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1168.

The film of water along the wall of a channel (or pipe) is not at rest, except along the surface of contact, even if the thickness is extremely slight. There is undoubtedly motion at an indefinitely short distance from the wall. The experiments in heating and hydraulics have both shown that films are thicker with lesser velocities, and that with certain very small but determinate velocities the entire contents of the pipe acts precisely like the film along the wall. The whole becomes a mass flowing somewhat coheringly yet plastically as a thick liquid (molasses) flowing through and out of a pipe. This is the case of pure viscous flow in which there are no eddies at all. It is just a smooth, relatively slow flow, in which, however, there is a transverse velocity gradient, or the velocity increases from nothing at the walls to a maximum at the center.

This transverse velocity gradient, expressed by the ratio, $\frac{d\,v}{d\,x}$, simply specifies the relative amount of sliding (shearing motion) per second of two parallel lamina of uniformly moving water, at a distance, $d\,x$, apart. The law of viscous friction states that the total shearing resistance, F, between these two neighboring lamina is $\mu \frac{d\,v}{d\,x} A$, in which, μ is the coefficient of viscosity; A, the area of either of the two lamina (the area along which the shear occurs); and $d\,x$ is indefinitely small. Thus, the intensity of shearing stress is:

$$\frac{F}{A} = \mu \frac{d\,v}{d\,x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Concerning the flow in a long straight, horizontal, cylindrical pipe, consider a co-axial cylinder (variable radius, x) of water in the pipe. The cylinder of water is being thrust forward by the difference in the pressures upon the two ends of the pipe. Then, the intensity of the shearing stress set up along the convex area of the co-axial cylinder of water is the difference of the end pressures, divided by the convex area of cylinder, or:

$$\frac{F}{A} = \frac{p\,\pi\,x^2}{2\,\pi\,x\,l} = \frac{p\,x}{2\,l} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Equating and integrating Equations (1) and (2): $-\mu \frac{d\,v}{d\,x} = \frac{p\,x}{2\,l}$, $v_c = \frac{p\,r^2}{4\,\mu\,l}$ and $v = \frac{p\,(r^2 - x^2)}{4\,\mu\,l}$, in which, v_c is the velocity along the axis (central) and v is the velocity at distance, x, from the axis.

Then, the total discharge and mean velocity of the water through the pipe are, respectively,

$$Q = \pi \int v\,(2\,x\,d\,x) = \frac{\pi\,p\,r^4}{8\,\mu\,l} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$V = \frac{Q}{\pi\,r^2} = \frac{p\,r^2}{8\,\mu\,l} \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

This analysis is substantially that given by Lamb* although it is less formal. Equation (3), thus established theoretically, is exactly that found experimentally by Poiseuille in 1840. Equation (4) demonstrates that the pressure drop is proportional to the velocity directly, and not to the square, for pure viscous flow.

The common idea seems to be that turbulent conditions have little to do with viscosity. Lamb states:†

> "The resistance, in the case of turbulent flow, is found to be sensibly independent of the temperature, and, therefore, of the viscosity of the fluid. This is what we should anticipate from considerations of dimensions, if it be assumed that R varies as the square of the mean velocity relative to the wetted surface."

Lamb also quotes formulas for the coefficient of ordinary skin friction as given by Unwin.‡

Notwithstanding the opinions and statements of these profound authorities, it is confidently submitted that all "frictional" resistance to flow is due, either directly or indirectly, to viscosity; and that, if the viscosity of the water were zero, it would be a frictionless fluid in which there could be no such resistances. Of course, there would be resistance to velocity changes, but that would relate merely to "conserved" energy, available to support the flow and would not represent "lost work", as in the case of frictional resistances. Hydraulic friction is due entirely to viscosity acting under one, or both, of two regimens: Direct-shearing; eddy-making and eddy-destroying.

How is it possible to pass from a "perfect" (frictionless) fluid to an actual ("viscous") fluid solely by introducing the character of viscosity, without having all frictional resistances dependent on the viscosity? If vortices could be generated in a perfect fluid at some definite relative rate, then energy would be absorbed by such eddies (that is, in their kinetic energy content) and lost work would result; but it is not apparent how such eddies could be generated in a perfect fluid, and it is very apparent that, if they were generated, there would be continually increasing turbulence progressing down stream, since a perfect fluid could not possibly dissipate any of its internal energy. Liquid motion cannot be "killed" without internal viscous resistances. Otherwise, there is no such thing as "impact" (of fluid upon fluid) in a fluid. Neither can there be impact upon a solid by a liquid in the sense that energy is destroyed by that impact. In every case it must be the property of viscosity which ultimately converts waste motion into heat. In fact, it seems only reasonable to conclude that the whole configuration and character of the eddy system is dependent to a great extent on viscosity, although, of course, the shape and character, as well as the boundaries, of the flow must have also a very great influence on the delivery of kinetic energy to the re-acting viscous resistances.

Consider now the turbulent flow of a river. The channel may be assumed to be uniform in section, that is, of a generally prismatic nature; but the wet perimeter is more or less undulatory and "rough". These undulations deflect water from the bottom and project it into the body of the stream, at greater or less velocity. The projected disturbances do not occur as definite streams of water, but as erratic pulsations, eddies, "boilers", and open vortices.

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* "Hydrodynamics," Third Edition, 1906, p. 543.

† *Loc. cit.*, p. 593.

‡ An error in the value of the coefficient, u, of viscosity given in the Ninth Edition of the Encyclopædia Britannica, was pointed out by the writer in 1909 in a paper on "Back-Water," *Minnesota Engineer*, Univ. of Minnesota. This error was carried forward in the Eleventh Edition of the Encyclopædia, and was again pointed out in a paper on "The Pitot Tube and Chemical Gauging," *Proceedings*, Engrs. Soc. of Western Pennsylvania, May, 1914.

In such a stream, a large deeply floating body, such as a ship, may float down stream either faster or slower (but usually faster), than the mean velocity of the water with which the body is in contact, depending on the distribution of velocities in the stream, the slope of the water surface, and the position, size, shape, and depth of flotation of the body.

In this connection Francis quotes Weisbach as follows: "As a rule * * * the velocity of a swimming body is somewhat greater than the water". This was investigated in an extensive study of the rod float, made by the writer, which is as yet (1929) unpublished.

Incidentally, the rod float is an excellent instrument for the measurement of the mean velocity of flowing water. It has been practically discarded by the profession, despite the fact that it is much more reliable and much simpler, for many purposes (even for refined work) than current meters, especially for gauging flow in canals of prismatic section. This simplification is made possible by the rod-float tables computed during the study.

It is important to note that the late J. P. Frizell, M. Am. Soc. C. E., actually calculated the theoretical velocity from that of a rod float for one condition of flow and depth of immersion. Francis distracted his attention, however, by showing that a float tended to slide down the inclined surface of the water and by stating that he thought it best to use empirical methods. Had it not been for this situation, Frizell would probably have solved the rod-float problem completely.

Theorem 1.—If the greater velocities (of an enveloping layer of water around a floating body, as a ship, or rod float) have the greater weights (averaging weights), and impulsive forces (due to skin friction) are proportional to the squares of impinging velocities, the velocity of the floating body is less than the mean velocity of the enveloping layer of water; but if the smaller velocities have the greater weights, the velocity of the floating body exceeds the mean velocity of the enveloping layer.

When the writer first demonstrated this theorem he was apparently squarely against Weisbach's statement that a ship would float down stream more rapidly than the current, because the theorem shows that a rod float will tend to move less rapidly than the water with which it is in contact. Nevertheless, as may be clearly seen, an application of the theorem to the two cases, taking into account the vertical velocity curve of the river (generally concave up streamward), and the size, shape, and position of the floating body, brings all into harmony.

If the law of skin friction relates to the first power of the impinging velocity only, then the floating body moves exactly with the mean velocity of the enveloping layer.

This theorem is independent of the tendency of the floating body to slide down the slope of the water surface, but another theorem shows that this tendency is generally of lesser importance.

Theorem 2.—If a body floats in a uniform current, and an independent propelling force acts upon that body in the line of action of the force due to hydraulic slope, then the total change, x, in velocity, from the condition in which the propelling force acts alone, to that in which the two forces combine, is less than the velocity which the hydraulic slope alone would produce, except the one case in which the propelling force is directed up stream and does not exceed the force due to slope.

The theorem does not except the case when the propelling force is directed up stream and exceeds the force due to slope. When the two forces are equal and opposed the change in velocity is, of course, just equal to the velocity due to slope.

The theorem is illustrated by the fact that a relatively small force will tow a large vessel in still water at a rate of several miles per hour, when the application of that same force in the line of motion when the vessel is being driven at full speed by its own power, would have scarcely any appreciable effect toward increasing, or decreasing, the speed. The theorem shows that the effect of slope is much diminished in the currents of rivers and canals, as against the action of slope in uniformly flowing water, because, in the latter case, there would be no independent propelling force.

These theorems are very important, because they have an illuminating relation to the questions of turbulent flow of rivers. Suppose, for example, a large mass of water is flowing down stream as a unit, with practically uniform velocity. The theory shows that it will tend to flow either faster, or slower, than the mean velocity of the water with which it is in contact. This would mean the generation of eddies and "boilers" around, and finally within, the body of water, so that there would be "internal" frictional losses within the body of water flowing in a stream due to the generation of turbulence internally.

This completes the picture of turbulent flow; it shows that these internal turbulent resistances may still be assumed to result from the effect of the bottom, or wet perimeter, because the variations of velocity must be due ultimately to disturbances emanating at and along the wetted surface.

To analyze this condition assume that the agitating area is the wet surface, measured by $P\,l$, in which, P is the perimeter of the cross-section, and l is the length of, or uniform reach of, the river. The volume of the water agitated along the wet surface is $n\,P\,l\,v$, in which, n is a dimensionless coefficient depending on the shape and character of the bottom, and v is the mean effective velocity with which the water is deflected from the agitating wet surface. If V is the actual mean velocity in the section, and A is the cross-sectional area:

$$n\,P\,l\,v = n\,\frac{P\,l\,v}{A\,V}\,Q = n\left(\frac{v}{V}\right)\frac{l}{m}\,Q \ldots\ldots\ldots\ldots (5)$$

since $\dfrac{P}{A} = \dfrac{1}{m}$, $Q = A\,V$ is the rate of discharge, and m is the hydraulic mean depth.

This volume of water deflected from the sides with effective velocity, v, has the kinetic energy,

$$\frac{n\,l\,v}{2\,m\,V}\,Q\,\delta\,v^2 \ldots\ldots\ldots\ldots (6)$$

All this is finally converted into heat by viscous resistances within the resulting eddies and the turbulence. It becomes lost as kinetic energy of the flow. The resulting loss of head is the loss of energy per pound of water discharged (total discharge); that is,

$$h = \frac{n\,v\,l}{2\,V\,m}\,\frac{Q\,\delta\,v^2}{w\,Q} = n\left(\frac{v}{V}\right)^3 \frac{l\,V^2}{m\,2\,g} \dots\dots\dots\dots\dots\dots (7)$$

Substituting for the quantity, $n\,\frac{v^3}{V^3}$, the factor, f, which is the dimensionless coefficient of loss due to turbulence, Equation (7) becomes:

$$h = f\,\frac{l}{m}\,\frac{V^2}{2\,g} \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots (8)$$

The ratio, $\frac{v}{V}$, denotes the ratio of the effective velocity of projection from the bottom to the mean velocity of flow in the river. Its value depends on the shape, dimensions, and other characteristics of the channel. The channel may be that of a pipe, but in that case f would be different. The reason such a dimensionless ratio can be merged with a dimensionless coefficient so as to form a modified coefficient, is that the mean velocity (as to time) at any given point in a river, in the case of steady flow, bears a constant ratio to the mean velocity of the river flow. This is a condition practically maintained in actual streams as regards mean point velocities in the cross-section, and this proposition is here extended to apply to the average pulsations in disturbances emanating from the wet area. Of course, the ratios change with change of stage, but, then, the dimensionless coefficients change also, so that no new character is attributed to the coefficients by the consolidation.

Observe carefully that Equation (8) is a rigorous theoretical formula except that, as yet, no method is shown for calculating the value of f. However, since the formula is the same as that used for years, values for f are available, based on a great amount of experience. As f is dimensionless, it has the same value in all systems of units, for any given channel. Observe, also, that the formula involves the exact square of the velocity, in so far as it relates to the losses of head resulting from pure turbulence. It is quite wrong to state that the loss due to turbulence (pure turbulence) varies in any other manner than as the exact square of the velocity (the mean velocity) in the cross-section. This is true, because all the velocities to which lost kinetic energy is to be assigned vary as the mean velocity in the cross-section, referring, of course, in each instance, to time averages for properly determined periods.

This discussion has treated separately the two cases—pure viscous flow and pure turbulent flow—each resulting in internal, as well as superficial losses along the wetted perimeter, the energy being dissipated in the form of heat. It is easy to understand that, along the boundaries in slowly moving pools as well as in the border films of turbulent flows, there must be more or less viscous resistance directed against the general flow. There must also be many places in the fluid where shearing resistances occur to check the flow, as it were. Thus, the total loss of head in friction is due to both causes acting simultaneously. Hence it is possible to add the two friction heads calculated in Equations (4) and (8). Since the pressure head of Equation (4) is $\frac{p}{w}$ (w being the specific weight of water):

$$h' = \frac{8\,\mu\,l\,V}{w\,r^2} = \frac{2\,\nu\,l\,V}{g\,m^2} = \frac{4\,\nu\,l\,V^2}{m\,V\,m\,2\,g} \ldots\ldots\ldots\ldots\ldots (9)$$

in which, h' is used to distinguish any directly effective viscosity resistances that may occur simultaneously with turbulent flow, but not including those viscous resistances which dissipate the turbulent energy by subduing it, so to speak.

Equation (9), however, is not yet fully adapted to be applicable simultaneously with turbulent flow. The velocity in Equation (9) was defined as that due to pure viscous flow. Hence, as in the case of the coefficient in Equation (8), to apply Equation (9) to the case in which both turbulent and viscous resistances act together, it is necessary to introduce a friction factor, f'. Thus, combining Equations (4) and (8):

$$H* = h' + h = \left(f' \frac{4\,\nu}{m\,V} + f\right) \frac{l\,V^2}{m\,2\,g} \ldots\ldots\ldots\ldots\ldots (10)$$

which is a perfectly sound formula for the flow of water in any channel. The ratio of pure stream-line velocity (which would be determined by Equation (4)) to the actual mean velocity was assumed to be absorbed in the coefficient, f', just as the ratio, $\frac{v^3}{V^3}$, was absorbed in f, in Equation (8).

Of course, f is not the same in Equation (10) as the usual friction factor. In Equation (10) both f' and f are dimensionless friction factors. They depend on the kinematical coefficient, ν, of viscosity; the mean velocity in the cross-section; the roughness of perimeter, including curvature; and the dimensionless ratios of channel dimensions, such as $\frac{l}{m}$, $\frac{l}{P}$, and $\frac{\varepsilon}{m}$, in which, ε is some linear measure of relative roughness; l, the length of reach; m, the hydraulic radius; and P, the bounding wet perimeter of cross-section.

In pure viscous flow, $f = 0$. In the case of turbulent flow the writer would not expect to find that f' has negligible values. In fact, it is perfectly clear that it cannot have such values, since it is well known in hydraulic experience that resistances in turbulent flow vary as a power of the velocity less than the square. When turbulence exists the coefficient of friction becomes:

$$F = f' \frac{4\,\nu}{m\,V} + f \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

and,

$$H* = F \frac{l}{m} \frac{V^2}{2\,g} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

It is clear from the value of F (which involves V in the denominator of one term) that H depends on a power of V less than the square, as is well known, but that there is no fixed relation of this kind applicable to all channels alike, and no fixed value for the exponent, even in the case of a given channel, although approximations have been made.* Equation (11) precludes the possibility of higher powers of V than the square, although, under extraordinary conditions, erroneous results may be obtained.

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* Forms for F in terms of definite negative fractional exponents of Reynolds' number relate to only one condition of roughness and one shape of channel boundary.

The correct solution of Equation (10) for the velocity is:

$$V = -\left(\phi\,\frac{\nu}{m}\right) + \sqrt{\left(\phi\,\frac{\nu}{m}\right)^2 + \left(\frac{m}{f\,l}\right)(2\,g\,H)}\,\ldots\ldots\ldots\ldots(13)$$

in which, $\phi = \left(2\,\frac{f'}{f}\right)$. The coefficients have been recast to adapt them to the form for the value of V.

Formulas of the general type of Equations (10) and (11), but without explicit mention of viscosity, have been proposed by eminent writers in the past. Biel's formula† takes into account the viscosity, but strict regard for dimensional form does not seem to have been followed, the idea apparently having been mainly to include two terms in metric units, one term in V, the other in V^2.

The idea of introducing ϕ in Equation (13) is to avoid obvious unnecessary mathematical difficulties by regarding ϕ and f as the only necessary coefficients. It should be noticed, however, that the coefficients of Equations (10) and (11) are properly the ones to be determined by experiment; but, for most practical purposes, the great amount of data available concerning the coefficient, F, in Equation (11), is quite sufficient.

Equation (13) includes Reynolds' critical velocity. When that occurs, f suddenly vanishes. By inserting the value of ϕ $\left(\text{that is, } \frac{2\,f'}{f}\right)$, it may be shown that Equation (13) reverts to Equation (9) for pure viscous flow. The velocity does not become infinite, as with some formulas.

The object of this discussion of hydraulic friction is not intended so much to adduce a new form of coefficient in terms of viscosity and turbulence, as to offer the true explanation of hydraulic resistance in channels of any kind, including weirs. The proposition that all hydraulic resistance depends ultimately on viscosity, so that, if this property were wanting, there would be no hydraulic resistance (lost work), is bound to meet with some incredulity, until those who think that turbulence resistance is independent of viscosity give the question careful investigation. The writer here takes the anomalous position that there can be no turbulence without viscosity; yet viscosity tends to suppress the initial formation of turbulence, as well as to still it after it has been formed.

As Equation (10) applies to any channel by dividing it into elementary longitudinal sections, it will apply to a weir, either with reference to the entire cross-section, or to the elementary filaments separately. It is clear that the loss of head in friction depends on all the variables on which the coefficients depend, and also on Reynolds' number. If $f = 0$ (no turbulence) the result is stream-line flow. In either kind of flow, the total coefficient decreases with an increasing Reynolds' number, $\frac{m\,V}{\nu}$, if f' and f do not change sufficiently to the contrary as seems usually to be the case. (Note the behavior in the vicinity of the critical value of Reynolds' number.)

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* Forms for F in terms of definite negative fractional exponents of Reynolds' number relate to only one condition of roughness and one shape of channel boundary.

† In 1909, the writer gave an account of Biel's formula in the *Minnesota Engineer*, Univ. of Minnesota, January, 1909.

Observe that Equation (10) measures merely the deterrent action of applied friction. That this friction is extremely small for weirs becomes apparent when the condition of a weir is compared with the condition of a canal, or river. Even with large values of the friction factors and with balancing slopes of only a few inches per mile, the velocities in rivers and canals are considerable. In the case of weirs the conditions are reversed, the surface slopes being far greater, while the friction is almost a vanishing quantity. This is due mainly to the small value of l, which attaches to weirs, in Equation (10), but the values of f' and f also become small, for weirs. However, it is well to consider the form of the coefficients in Equation (13).

Almost the entire obstacle to the surface slope and velocity over weirs lies in the acceleration of the spilling water above and beyond the crest. A frictionless weir would deliver only a little more water. Consequently, hydraulic friction is of little or no account in its effects on weir discharge, unless the head is abnormally small or the kinematical viscosity abnormally large. That the acceleration at Keokuk is very important, especially in furnishing a gauging section free from turbulence, is mentioned by the authors. In such a section the effect of any turbulence near the piers must be extremely small, relatively.

In preparing their paper, and in executing their tests, the authors have been careful not to say or do anything which will admit of any great criticism, so that, actually, there is not much to criticize. However, it may be pertinent to comment favorably on some of their statements. Perhaps the main value of the experiments, other than the great practical utility of a ready means for determining exact coefficients for weirs of the Keokuk type, and approximate coefficients for other types, is the information concerning the interior of the nappe. It would have been of great value also to determine internal pressures, but, if this was not done, the writer knows well that one is not usually warranted in going far beyond the immediate objects of experiment to accumulate data on concomitant phenomena.

For many practical purposes Fig. 14 will be all that is desired, but there is need for a true weir formula, especially as an aid in original design. Much of what the authors have done will be useful in framing such a fundamental theory. It is true that there are some remarkable studies in hydrodynamics, but, when it comes to weirs, the theory is lacking, even for the case of two-dimensional flow. The remarks of the authors are, therefore, of very great interest.

They state four significant facts, as follows: (1) Velocities are remarkably uniform within the nappe at a given depth below the pond level; (2) velocities within the nappe increase downward and friction is very small; (3) at the entrance to the nappe, velocities are higher at the bend around the pier noses;

and (4) velocities at the entrance to the nappe are retarded by the restricted, and relatively constricted, paths along the angular corners.

The opinion may be expressed that one reason for the increase in velocity within the nappe, passing downward, lies in the fact that the spillway crest is curved in a vertical plane, and the radius is directed downward toward the center of curvature. Thus, the water in its progress is given an opportunity to turn downward, so that the centripetal acceleration, $\frac{v^2}{r}$, produces an upward directed centrifugal reaction which tends to support the weight of the water above, thus causing a decrease of internal pressure and a resulting increase in velocity, passing downward when comparing conditions. The reverse will be the case in the depression over an apron at the foot of an ogee dam. Velocities will be higher at the surface than at a depth, while the pressure will increase downward, because the concavity of the water surface faces upward. This is exactly the effect obtaining in an eddy with vertical axis (for example, water rotating rapidly in a pail), the difference being that the axis of rotation over the weir is horizontal. This same logic explains the increase in the velocity around the bends at the noses of the piers. The effect is the result of decreased pressure toward the center of curvature approximately according to the formula,

$$\frac{d\,p}{d\,r} = \frac{w}{g}\,\frac{v^2}{r} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Equation (14) would be exact if the viscosity were zero; that is, when there are no tangential tractions acting on elementary surfaces along which there is a tendency to slide. In hydrodynamics, when there are no such tractions, it has been assumed that v varies inversely as r, which leads to the "irrotational" form of vortex. There is another more or less present cause for reduced pressure on the crest and down along the spillway; that is, the siphonic action of the column of water resting on and against the spillway.

With reference to Fig. 14, it was suggested that possibly the velocity at the surface of the nappe may exceed that due to the head below pond level on the surface point in question. The possible shearing traction exerted by the more rapidly moving water below seems plausible, as a cause, at first sight, but the fact that the excess velocity is negligible, compared to what one might expect, raises some doubt. One may imagine a weir discharging a very viscous liquid, say, semi-liquid tar. With that in mind there does not appear to be any real connection between actual velocities and the broken-line curve of "Velocity Corresponding to Head below Pond Level".

Relative to the Keokuk weir, it may be pertinent to remark that the writer made some study of the plans for that weir and the possible coefficient of discharge, long before it was built. A number of engineers then entertained the idea that the adopted weir coefficients were too small for that type.

The writer's experience on the St. Lawrence and many other large rivers leads him to believe that, if the modeled reach is not too long (say, a mile or two), models using water give surprisingly accurate reproductions of flow. Perhaps longer reaches might prove satisfactory. He does not doubt the approximate accuracy of a model of the Keokuk spillway to a scale ratio of 1 to 100.

However, the writer has shown* that a mercury "model" of water about one-fifth the full size will give theoretically exact results. That being true, it should be possible to secure the same degree of accuracy with a mercury model one–five hundredth of the full size as with a water "model", one-one hundredth of the full size. One should not hesitate to test and rely on a mercury model of the Keokuk spillway, one-two hundred and fiftieth of the full size, although it would have to be "built like a watch". Such a model spillway must be built of suitable materials in proper condition as to smoothness, etc., in order to avoid distortion caused by excess viscosity and the presence of surface tension. An application of model testing to large structures has been fully described by the writer under the title, "Ice Diversion Works for St. Lawrence River Power Company."†

In closing, the writer may state that he can confirm the necessity of "staying" the meters in swift water. He has used meters in the rapids of the St. Lawrence where there were velocities approximating, if not as great as, those experienced at Keokuk by the authors.

As to the weir, the writer has made some calculations for the internal pressures at several points within the nappe (approximately, at least) and believes that it would be quite possible to frame a true weir formula, basing practical coefficients (dimensionless) on the results presented in the paper.

FLOYD A. NAGLER,‡ AND ALBION DAVIS,§ MEMBERS, AM. SOC. C. E. (by letter).—The writers are deeply appreciative of the constructive comment made by the various discussers. Apparently, little remains to be said in closing the discussion. It is, indeed, gratifying to note the close agreement between the coefficients obtained on the Keokuk model and those presented by Dr. Ott, resulting from experiments at the Research Institute at Munich. The writers' estimate that their experiments were reliable within 2% was not based upon the laboratory verification by means of the Pitot tube alone; it was a judgment based upon the general reliability of the current-meter methods and ratings used in the field, together with the various checks which were afforded by other measurements and the model tests.

Additional data were requested§ relative to the behavior of various types of current meters in streams with varying degrees of angularity, both from the left and the right. The average corrections applied in the paper were based on angular calibrations made at the U. S. Bureau of Standards. However, these were subsequently checked in an extensive investigation‖ at the laboratory of the University of Iowa, made in conjunction with the Bureau of Public Roads, U. S. Department of Agriculture.‖

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* "Ice Diversion, Hydraulic Models, and Hydraulic Similarity", *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1187, footnote.

† *Canadian Engineer,* November 25, 1920.

‡ Prof., Hydr. Eng., Univ. of Iowa, Iowa City, Iowa.

§ Chf. Engr., Mississippi River Power Co., Keokuk, Iowa.

‖ "Effect of Turbulence on the Registration of Current Meters", by David L. Yarnell and Floyd A. Nagler, Members, Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2611.

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Paper No. 1745

ELASTIC EQUILIBRIUM IN THE THEORY OF STRUCTURES*

By H. S. Richmond,† Esq.

With Discussion by Messrs. Jacob Feld, F. B. Seely, W. J. Putnam, and W. L. Schwalbe, Norman Litchfield, E. R. Hill, O. G. Julian and George R. Rich, John I. Parcel, S. Timoshenko, and H. S. Richmond.

Synopsis

The standard specifications and formulas used in structural design, were devised to meet the problems encountered in ordinary bridge and building construction. They are well adapted to this purpose and have been found convenient and practical. When, however, an attempt is made to treat certain special problems, the limitations of these standard formulas become apparent. For instance, in the reinforcement of existing structures, an investigation of their strength depends on a rational analysis of their loads, stresses, and strains, and the design of their reinforcement must be based on such analysis, rather than on any empirical formulas.

Another class of structure which demands special treatment is that used in railroad electrification work for the support of transmission wires, trolley catenary wiring, outdoor sub-station wiring, and switching. For some years, the writer has been actively engaged in the design of such structures, and has found it entirely practical to use bents which, while thoroughly braced, neither follow the usual design nor stay within the limits set up in some of the conventional rules.

As a general rule members have been of simple and compact form and of a smooth surface, because, from the standpoint of maintenance, it is important

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* Published in January, 1929, *Proceedings*.

† With Gibbs & Hill, Cons. Engrs., New York, N. Y.

to avoid lattice-bars and other intricate details. This is especially true of structures such as catenary-supporting bridges which span the railroad tracks and are exposed to exhaust gases from steam locomotive stacks.

This has finally led to the adoption of what is virtually a single-member structure; that is, two legs and one cross-beam or wire-span, instead of square latticed columns and a cross-box girder, or four-post structure. The entire structure lies almost within one vertical plane, with a minimum of lateral bracing. Consequently, the fundamental mechanical principles applying to light and slender members must be carefully analyzed in the development of a safe and economical design.

In addition to economy, structures so designed have the advantage (when used for supporting wires which are strung under considerable tension) that, in the event of a failure of one or more wires, the structure flexes a considerable amount without destructive effect, distributing the emergency load among a number of bridges.

Light structures of the type used for cross-country transmission towers, etc., are quite generally built of standard rolled sections, with slender members of thin material, and must necessarily have many unsymmetrical and eccentric connections and few rivets or bolts. In addition to the specified loads which they are designed to carry, such structures must sustain other loadings, such as their own weight, wind pressure, the weight of a man climbing the tower, etc. If standard formulas are used in the design, a member would be obtained unquestionably strong enough so that it would not be necessary to consider these secondary loadings, but the design would be uneconomical. When it is understood that the cost of transporting these towers to the site and erecting them may amount to a considerable addition to the cost of their purchase, it will be found profitable to apply an analytical treatment, especially where there is much duplication of type.

The foregoing applies chiefly to structures having light members. Quite different considerations apply to heavy structural members, for which the engineer is justified in giving a very careful study to the economics of design with a view of obtaining a member of more nearly ideal proportions. For example, in a cantilever bridge of great span, the dead load is often much greater than the live load. The saving of a pound weight of steel at a point in mid-span will reduce the dead load and save many pounds elsewhere in the structure. In such a case the use of high-strength alloys appeals to the imagination, but in utilizing such alloys, it is necessary to consider that the elastic modulus of steels has not kept pace with their elastic limit and ultimate strength. If a higher compressive stress is to be used, especial care must be given to the consideration of elastic stability or resistance to various kinds of crippling. A long column may fail by bowing or by a wrinkling of the plates and flanges which form its section. The make-up of a column is governed by a balancing of these various points of consideration, and the use of a steel other than the standard involves a fresh analytical treatment.

A thorough study of the column should include that of the resistance of its elements to failure by any modes which may be classified under the heading of neutral elastic equilibrium. Its web-plates must not be thin enough to fail by wrinkling; its flanges must not crimp; and if it is a single-angle member it must have sufficient torsional rigidity to prevent failure by twisting.

This principle of elastic equilibrium is given consideration in structural design and special attention is given to its consideration in connection with the design of columns, to which a great amount of study has been devoted and on which subject the literature is voluminous.

While the column has been thus honored, other structural members, such as beams, have been given less consideration. The writer published a small pamphlet in 1912, entitled "The Lateral Deflection of Beams", in which he set forth the results of some experiences with model beams and offered an analytical treatment of the problem of the elastic equilibrium of beams. This, with the exception of a study by Dr. S. Timoshenko,* is, in so far as he knows, the only analytical treatment of this problem.

Two distinct classes of beams were recognized, the flanged beam without torsional resistance and the beam of compact form, but with torsional rigidity. Subsequently, the theorems were extended to apply to a flanged beam having torsional resistance, and to include channel-shaped as well as symmetrical sections. Tests of small wooden model beams showed a satisfactory conformity with the theorem for the class tested, namely, the beam with considerable torsional resistance.

NOTATION

The following notation has been used in the paper:

a = distance (measured in the plane of the web) of the point of application of the load above the center of the beam.
$-a$ = distance (measured in the plane of the web) of the point of application of the load below the center of the beam.
b = distance from center to center of channel or I-beam flanges.
f = fiber stress.
h = distance of the fiber, f, from an axis that is perpendicular to the web, through its center.
k_1 = lateral curvature of the upper flange of a beam.
k_2 = lateral curvature of the lower flange of a beam.
n = eccentricity of the applied load (see Fig. 4).
r = modulus of rigidity $= \dfrac{Q}{\alpha}$.
S = shear.
x = abscissa of a point on a beam, measured in a horizontal plane from the Y-axis.
y = ordinate of a point, the abscissa of which is x.
z = abscissa of a point, the ordinate of which is h.
A, B, C, and D = arbitrary constants.
E = modulus of elasticity.
E_s = shearing modulus of elasticity.
I = moment of inertia about the originally horizontal gravity axis.
I' = least moment of inertia.
I_h = moment of inertia (about a horizontal axis) of a half section of a beam.

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* "Beams Without Lateral Support," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 1247.

I_1 = moment of inertia of the section about its originally vertical gravity axis.

$I_3 = \dfrac{I_1}{1 - \dfrac{I_1}{I}}$.

I_z = moment of inertia of the upper half section of a beam about its originally vertical axis through the center of the web.

J = factor expressing the capacity of any cross-section to resist torsion.

K = factor $= T L^2 = \dfrac{P L^2}{\sqrt{E I_3 E_s J}}$.

L = length of a cantilever beam or column; or one-half the length of a beam supported at each end.

M = applied bending moment in a beam and parallel to the web.

M_2 = bending moment in a beam induced by torsion, and perpendicular to the web.

M_3 = bending moment in the flanges of a beam induced by torsion, and parallel to the web; $+ M_3$ applies to upper flange, and $- M_3$ to lower flange.

M_t = lateral bending moment in the upper flange of a beam.

M_b = lateral bending moment in the lower flange of a beam.

N = factor $= \dfrac{a}{L} \sqrt{\dfrac{E I_3}{E_s J}}$.

P = compressive load or the load necessary for neutral elastic equilibrium.

Q = torque due to the applied load, P.

R = factor.

T = factor such that $T^2 = \dfrac{P^2}{E I_3 E_s J}$.

U = $\int\int hz\, dh\, dz$ for the upper half section referred to the central point of the web.

$\alpha = \dfrac{Q}{r}$ = angle of deflection of the web due to torque, Q (see Fig. 3).

δ, ε, and λ = factors.

Δ = horizontal displacement of a load, P, producing torsion in a beam.

DEFINITIONS

The term, "elastic equilibrium", as herein used, is to be understood as meaning the state of an elastic body (under a condition of stress resulting from the application of an external loading) with respect to its ability to resist excessive deformation or change of form initiated by a second, or disturbing, force temporarily applied. A body is in stable equilibrium when it has a tendency to regain its original form on removal of the disturbing force.

Likewise, neutral elastic equilibrium exists when the body has an equal tendency to regain its original form, to remain in its deformed condition, or to increase its deformation after removal of the disturbing force.

The "pin-ended" Euler column furnishes a familiar example illustrating this principle. In this case the column is in neutral equilibrium when $P = \dfrac{\pi^2 E I'}{L^2}$.

If P is less than this amount, the column is in stable equilibrium and if it is greater, the column is in unstable equilibrium.

Consider a horizontal shaft of length, L (Fig. 1(a)), fixed at one end (the other end supported but free to rotate), and carrying a rigid arm extending vertically upward and sustaining a load acting vertically downward at its upper extremity. Suppose that the shaft undergoes a torsional deflection, α, from any cause, so that the arm at the end assumes an inclined position as shown in Fig. 1(b). From Fig. 1,

$$Q = P\,\Delta = P\,a\,\alpha \quad \text{.......................(1)}$$

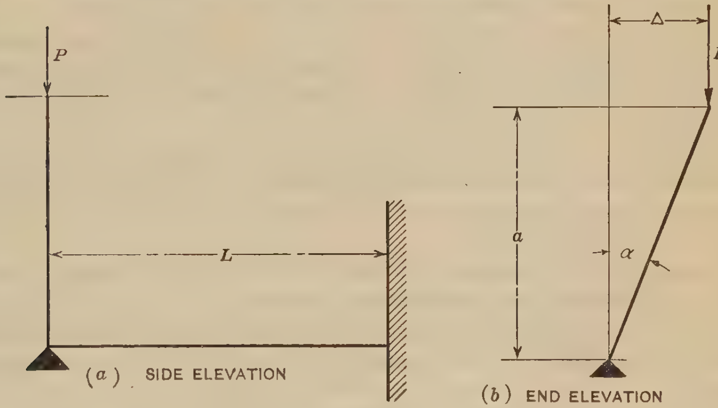
and since, by definition, $\alpha = \dfrac{Q}{r}$,

$$P = \frac{r}{a} \quad \text{...........................(2)}$$

which is the relation sought.

Let J equal a factor expressing the capacity of any cross-section to resist torsion, so that $\alpha = \dfrac{Q\,L}{E_s\,J}$; then $r = \dfrac{E_s\,J}{L}$, and,

$$P = \frac{E_s\,J}{a\,L} \quad \text{...........................(3)}$$



(a) SIDE ELEVATION (b) END ELEVATION

FIG. 1.

In practice, all structures must meet the condition of stable elastic equilibrium and their loadings must not approach too closely the loadings which would result in neutral elastic equilibrium. Even so, account must be taken of the results of deformation under load, as affecting stress conditions. This can be accomplished in two ways: (1) The problem of the particular structural member, which is in a state of deformation and, at the same time, is in stable elastic equilibrium, may be analyzed; or (2) the designer's judgment may be governed by the following considerations:

(a) A member subject to deformation due to any system of loading (the primary loading), will in general be less well adapted to resist the effects of another system, (the secondary loading), than if the first system of loading did not exist.

(*b*) If the primary system of loading does not exist, then the entire strength of the member can be devoted to the resistance of the secondary system.

(*c*) If the primary system of loading is such as to place the member in a state of neutral elastic equilibrium, then the member has generally no strength to resist the secondary system, but is, so to speak, in a state of ruin or uselessness as a structural member.

Between the limits of no primary load and a primary load resulting in neutral elastic equilibrium, it may be shown that for some members, loaded in a certain way, the load-resisting function of the member as regards the secondary system is approximately proportional to 1 minus the ratio of the primary load or system to a similar load or system which would result in a condition of neutral elastic equilibrium. This relation forms the basis of some useful formulas.

In the absence of analysis, certain problems may be treated in this manner, but it should be noted that the relation is by no means generally applicable and must be used judiciously.

Columns having considerable eccentricity of loading and members subject to combined bending and compression may be treated conveniently and rationally by formulas based on Considerations (*a*), (*b*), and (*c*). The remainder of this paper, however, will be devoted largely to an exposition of the theory of elastic equilibrium of beams, leading to the expression of formulas that indicate the value of loadings at which beams will fail by lateral deflection.

A slender beam of certain proportion will fail by lateral deflection in a manner analogous to the failure of a slender column. It may be shown, however, that an attempt to treat beams by formulas familiar in column design will lead to erroneous results.

For convenience in analysis, the writer divides beams roughly into two classes: (1) "J-beams", in which torsional resistance plays an important part in providing lateral stability; and (2) "F-beams" which are of I-section, or channel section, in which the value of J is so small that its influence is negligible.

Many beams do not conform to these pure types, but share the characteristics of each. Channel-shaped beams are a peculiar type and their analysis is extended.

Proper Use of Channel Members as Beams

In a channel consider a system of external forces transverse to its length, acting in the plane of the web and perpendicular to the axis of the channel. The resulting bending will produce longitudinal tensile and compressive stresses in the flanges, but these stresses will be greater at the heel than at the toe of the flange. It is easy to imagine this, since the stresses result from the shear in the web which acts directly on the heel and not on the center of the flange as in the case of I-beams.

In the analysis it is assumed that the fiber stress in any given cross-section of the channel varies uniformly from heel to heel of the two flanges and also from heel to toe of each flange. This is more conservative than the usual assumption that the stress varies uniformly throughout the section and in proportion to the distance of the point considered from an axis of the section. The analysis shows the latter assumption to be erroneous as applied to the channel.

If the influence of J is neglected, then knowing the bending moment applied in a direction parallel to the web, it would be simple to compute the fiber stresses in the flange, and the stress at the toe would be of opposite sign to that at the heel. The influence of J, however, is to make the unit stress more nearly equal, throughout the section of the flange, especially as the beam increases in length. On the other hand, as the beam becomes very long, the slenderness effect becomes of greater importance and must be considered in the design.

It is possible to apply the external loads so as to make the distribution of flange stress uniform from heel to toe, and in this case the fiber stress at any point would be governed by the formula, $f = \frac{M\,h}{2\,I_h}$.

This ideal position of the loading system is that of a plane which lies parallel to the web and a distance back from the center of the web equal to $\frac{U\,b}{2\,I_h}$, as shown in Fig. 2.

Safe design will then provide that this relation be adhered to closely, or else that the stresses be analyzed and considered. Furthermore, the condition of neutral elastic equilibrium should not be allowed. This applies only to a beam which is not restrained laterally so as to prevent torsional and lateral deformation.

NEUTRAL ELASTIC EQUILIBRIUM OF THE J-BEAM

As treated under this heading, J-beams are assumed to be of uniform, compact cross-section, symmetrical about each principal axis, and capable of acting in pure torsion. The beam is more slender as regards that principal axis of inertia which lies in the plane of the applied loading. It is assumed that the cross-sectional form of the beam is conserved under conditions of stress and that there is a property adhering to the central axis of the beam which represents its ability to resist torsional deformation and which is symbolized by the letter, J.

In Fig. 3, let the curved line represent the horizontal projection of the central axis of a cantilever beam which has been deflected laterally.

At its right end, it is fixed, its axis is horizontal, and the two axes of symmetry of its cross-section are horizontal and vertical, respectively. Its moment of inertia about the horizontal axis is the greater. The beam is weightless and carries a load, P, acting vertically downward, and applied at a point, a distance, a, above the central axis of the beam at its left end. The beam suffers both bending and torsional strains, and is in a state of neutral elastic equilibrium.

At any section, x, y, its normally vertical axis is inclined an angle, α, with the vertical. It is required to find an expression for P, the load necessary to produce neutral elastic equilibrium. The origin of co-ordinates is taken at the horizontal projection of the point of loading. On the left of Fig. 3 are shown an end view and two cross-sections. As plotted in the diagram, α and $\frac{d\,y^2}{d\,x^2}$ are negative and $\frac{d\,\alpha}{d\,x}$, $\frac{d\,y}{d\,x}$, and y are positive.

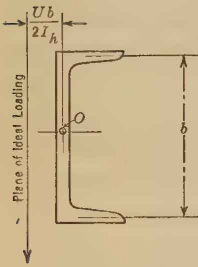


FIG. 2.

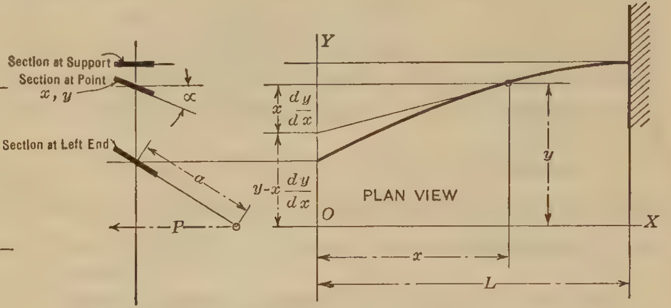


FIG. 3.

Let J be a function of the section representing its ability to resist torsional deformation, and of such a value that, disregarding the resistance of the flanges,

$$d\,\alpha = \frac{Q\,d\,x}{E_s\,J} \quad \text{(4)}$$

The bending moment at any point is $M = Px$. The torque, Q, at any point is equal to the load, P, times the distance from its line of action to a tangent to the central axis of the beam at the point considered; that is (see Fig. 3):

$$Q = P\left(y - x\frac{d\,y}{d\,x}\right) \quad \text{(5)}$$

The component of the bending moment in a direction perpendicular to its originally vertical axis is $Px \sin\alpha$ and, in a direction at right angles to this direction, is $Px \cos\alpha$.

The components of the curvature in each direction are $\frac{P\,x \sin\alpha}{E\,I_1}$ and $\frac{P\,x \cos\alpha}{E\,I}$, respectively.

The curvature of the horizontal projection of the axis is obtained by multiplying these expressions by the cosine and sine, respectively, of the angle of inclination, and taking the difference of the results, or

$$\frac{d^2\,y}{d\,x^2} = \frac{P\,x \sin\alpha \cos\alpha}{E\,I_1} - \frac{P\,x \cos\alpha \sin\alpha}{E\,I}$$

or, since α is small,

$$\frac{d^2\,y}{d\,x^2} = \frac{P}{E}\left(\frac{1}{I_1} - \frac{1}{I}\right) x\,\alpha = \frac{P}{E\,I_3}\,x\,\alpha \quad \text{(6)}$$

From the law of torsional deflection,

$$\frac{d\,\alpha}{d\,x} = \frac{P}{E_s\,J}\left(y - x\,\frac{d\,y}{d\,x}\right) \quad \ldots\ldots(7)$$

Solving for α in Equation (6),

$$\alpha = \frac{E\,I_3}{P}\,\frac{1}{x}\,\frac{d^2\,y}{d\,x^2} \quad \ldots\ldots(8)$$

Differentiating Equation (8),

$$\frac{d\,\alpha}{d\,x} = \frac{E\,I_3}{P}\,\frac{1}{x}\,\frac{d^3\,y}{d\,x^3} - \frac{E\,I_3}{P}\,\frac{1}{x^2}\,\frac{d^2\,y}{d\,x^2} \quad \ldots\ldots(9)$$

Equating Equations (7) and (9), clearing of fractions and substituting T^2 for its equivalent, $\frac{P^2}{E\,I_3\,E_s\,J}$,

$$x\,\frac{d^3\,y}{d\,x^3} - \frac{d^2\,y}{d\,x^2} + T^2\,x^3\,\frac{d\,y}{d\,x} - T^2\,x^2\,y = 0 \quad \ldots\ldots(10)$$

The solution of Equation (10) is,

$$\begin{aligned} y = {} & A\left(1 + \frac{T^2\,x^4}{12} - \frac{T^4\,x^8}{4\times5\times7\times8} + \frac{T^6\,x^{12}}{4\times5\times8\times9\times11\times12} - \ldots\right) \\ & + B\,x + C\left(x^3 - \frac{T^2\,x^7}{2\times6\times7} + \frac{T^4\,x^{11}}{2\times7\times8\times10\times11}\right. \\ & \left. - \frac{T^6\,x^{15}}{2\times7\times8\times11\times12\times14\times15} + \ldots\right) \quad \ldots\ldots(11) \end{aligned}$$

Differentiating Equation (11) successively,

$$\begin{aligned} \frac{d^2\,y}{d\,x^2} = {} & A\left(T^2\,x^2 - \frac{T^4\,x^6}{20} + \frac{T^6\,x^{10}}{1\,440} - \ldots\right) \\ & + C\left(6\,x - \frac{T^2\,x^5}{2} + \frac{T^4\,x^9}{112} - \frac{T^6\,x^{13}}{14\,784} + \ldots\ldots\right) \quad \ldots\ldots(12) \end{aligned}$$

From the condition, $y = -a\,\alpha$, when $x = 0$, and from Equations (11), (8), and (12),

$$A = -a\,\alpha = -\frac{6\,E\,I_3\,a\,C}{P} \quad \ldots\ldots(13)$$

From the condition, $\alpha = 0$ when $x = L$, and from Equations (8) and (12),

$$\begin{aligned} 0 = {} & \frac{E\,I_3\,A}{P\,L^3}\left(K^2 - \frac{K^4}{20} + \frac{K^6}{1\,440} - \ldots\right) \\ & + \frac{E\,I_3\,C}{P}\left(6 - \frac{K^2}{2} + \frac{K^4}{112} - \frac{K^6}{14\,784} + \ldots\ldots\right) \quad \ldots\ldots(14) \end{aligned}$$

Eliminating A and C between Equations (13) and (14),

$$\begin{aligned} 0 = {} & -\frac{6\,E\,I_3\,a}{P\,L^3}\left(K^2 - \frac{K^4}{20} + \frac{K^6}{1\,440} - \ldots\ldots\right) \\ & + \left(6 - \frac{K^2}{2} + \frac{K^4}{112} - \frac{K^6}{14\,784} + \ldots\ldots\right) \quad \ldots\ldots(15) \end{aligned}$$

Since $N = \frac{a}{L}\sqrt{\frac{E\,I_3}{E_s\,J}}$, $P = T\sqrt{E\,I_3\,E_s\,J}$, and $T\,L^2 = K$, therefore,

$$\frac{E\,I_3\,a}{P\,L^3} = \frac{E\,I_3\,a}{T\,L^3\sqrt{E\,I_3\,E_s\,J}} = \frac{a}{T\,L^3}\sqrt{\frac{E\,I_3}{E_s\,J}} = \frac{a}{L\,K}\sqrt{\frac{E\,I_3}{E_s\,J}} = \frac{N}{K}$$

Substituting these values in Equation (15),

$$0 = -6N\left(K - \frac{K^3}{20} + \frac{K^5}{1\,440} - \ldots\right) + \left(6 - \frac{K^2}{2} + \frac{K^4}{112} - \frac{K^6}{14\,784} + \ldots\right) \ldots\ldots (16)$$

$$N = \frac{6 - \frac{K^2}{2} + \frac{K^4}{112} - \frac{K^6}{14\,784} +}{6\left(K - \frac{K^3}{20} + \frac{K^5}{1\,440} - \ldots\right)} \ldots\ldots (17)$$

$$N = \frac{1}{K} - \frac{K}{30} - \frac{11\,K^3}{12\,600} - \ldots\ldots (18)$$

Equation (18) holds for both positive and negative values of N. K occurs only in the odd powers; therefore, changing the sign of K changes the sign of N without otherwise affecting its value.

When $N = 0$, the numerator of the right-hand member of Equation (17) becomes 0, and the lowest root of K^2 is 16.11, or,

$$K = T\,L^2 = \frac{P\,L^2}{\sqrt{E\,I_3\,E_s\,J}} = \pm\,4.014$$

or,

$$P\,L^2 = 4.014\,\sqrt{E\,I_3\,E_s\,J}$$

When $N = -\infty$, the denominator of the right-hand member of Equation (17) is 0; then,

$$K = +\,5.6$$

or,

$$P\,L^2 = 5.6\,\sqrt{E\,I_3\,E_s\,J}$$

For large negative values of N, K approaches 5.6 and for large positive values of N, K approaches $\frac{1}{N}$, or $\frac{P\,L^2}{\sqrt{E\,I_3\,E_s\,J}}$ approaches $\frac{L}{a}\sqrt{\frac{E_s\,J}{E\,I_3}}$, or P approaches $\frac{E_s\,J}{a\,L}$. Compare this with the value of P in Equation (3). The simple beam may be treated in a similar manner.

NEUTRAL ELASTIC EQUILIBRIUM OF THE FLANGED BEAM

In this case an I-beam having no torsional resistance is assumed. The resistance of the flanges to lateral flexure is assumed to be located at their centers.

Let the cantilever I-beam shown in Fig. 4 be assumed as fixed at the right end, both flanges being restrained at this end against lateral rotation. The beam is shown in a deflected position. Its originally vertical web is tilted at the end as shown. It supports a load, P, at the end, applied a distance, a, above the center of the beam, and a distance, n, horizontally from the center line of the web.

At any point (x, y), the beam is under the following conditions of stress: A moment, M_3, in the upper flange and $-M_3$ in the lower; a moment, M_2, in the beam and perpendicular to the web; and a moment, M, in the beam and parallel to the web.

From Fig. 4, $y = \frac{y_1 + y_2}{2}$; $\alpha = \frac{y_1 - y_2}{b}$; $\frac{d\,\alpha}{d\,x} = \frac{1}{b}\left(\frac{d\,y_1}{d\,x} - \frac{d\,y_2}{d\,x}\right)$; $M_1 = P\,x$; and $M_2 = P\,x\,\alpha = M\,\alpha$. Then,

$$\frac{d\,M_t}{d\,x} = \frac{P}{b}\left(x\frac{d\,y_1}{d\,x} - y_2\right) \qquad (19)$$

$$\frac{d\,M_b}{d\,x} = \frac{P}{b}\left(-x\frac{d\,y_2}{d\,x} + y_1\right) \qquad (20)$$

$$\frac{d\,M_3}{d\,x} = \frac{\frac{d\,M_t}{d\,x} - \frac{d\,M_b}{d\,x}}{2} = \frac{P}{2\,b}\left[x\left(\frac{d\,y_1}{d\,x} + \frac{d\,y_2}{d\,x}\right) - (y_2 + y_1)\right]$$

$$= \frac{P}{b}\left(x\frac{d\,y}{d\,x} - y\right) \qquad (21)$$

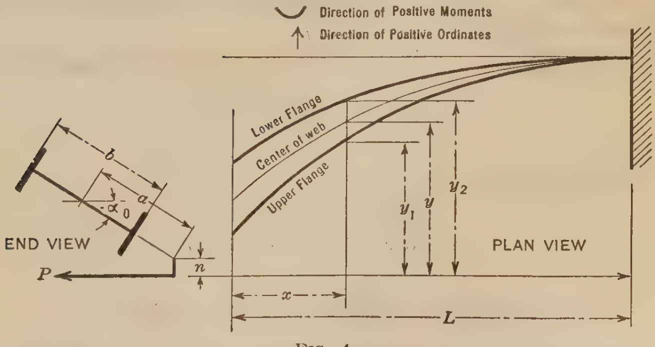
$$\frac{d^2\,\alpha}{d\,x^2} = \frac{k_1 - k_2}{b} = -\frac{2\,M_3}{E\,b\,I_z}, \text{ or, } M_3 = \frac{E\,b\,I_z}{2}\frac{d^2\,\alpha}{d\,x^2} \qquad (22)$$

$$\frac{d\,M_3}{d\,x} = \frac{E\,b\,I_z}{2}\frac{d^3\,\alpha}{d\,x^3} \qquad (23)$$

$$\frac{d^2\,M_3}{d\,x^2} = \frac{E\,b\,I_z}{2}\frac{d^4\,\alpha}{d\,x^4} = \frac{P}{b}\frac{d^2\,y}{d\,x^2} \qquad (24)$$

$$\frac{d^2\,y}{d\,x^2} = \frac{P\,x\,\alpha}{2\,E\,I_z} - \frac{P\,x\,\alpha}{2\,E\,I_h} = \frac{P\,x\,\alpha}{2\,E}\left[\frac{1}{I_z} - \frac{1}{I_h}\right] \qquad (25)$$

$$\frac{E\,b\,I_z}{2}\frac{d^4\,\alpha}{d\,x^4} = \frac{P^2\,x^2\,\alpha}{2\,E\,b}\left[\frac{1}{I_z} - \frac{1}{I_h}\right] \qquad (26)$$



Direction of Positive Moments
Direction of Positive Ordinates
Lower Flange
Center of web
Upper Flange
b
α
α0
END VIEW
P
n
y1
y
y2
PLAN VIEW
x
L

FIG. 4.

or,

$$\frac{d^4\,\alpha}{d\,x^4} - R^2\,x^2\,\alpha = 0 \qquad (27)$$

in which,

$$R^2 = \frac{P^2}{E^2\, b^2\, I_z}\left[\frac{1}{I_z} - \frac{1}{I_h}\right]$$

The solution of Equation (27) is,

$$\alpha = A\left[\frac{1}{2} + \frac{(R\,x^3)^2}{\underline{\lfloor 6}} + \frac{56\,(R\,x^3)^4}{\underline{\lfloor 12}} + \frac{10\,192\,(R\,x^3)^6}{\underline{\lfloor 18}} + \ldots\ldots\ldots\ldots\right]$$

$$+ B\,x\left[\frac{1}{6} + \frac{(R\,x^3)^2}{\underline{\lfloor 7}} + \frac{72\,(R\,x^3)^4}{\underline{\lfloor 13}} + \frac{15\,120\,(R\,x^3)^6}{\underline{\lfloor 19}} + \ldots\ldots\ldots\right]$$

$$+ C\,x^2\left[\frac{1}{2} + \frac{12\,(R\,x^3)^2}{\underline{\lfloor 8}} + \frac{1\,080\,(R\,x^3)^4}{\underline{\lfloor 14}} + \frac{259\,200\,(R\,x^3)^6}{\underline{\lfloor 20}} + \ldots\right]$$

$$+ D\,x^3\left[\frac{1}{6} + \frac{20\,(R\,x^3)^2}{\underline{\lfloor 9}} + \frac{2\,200\,(R\,x^3)^4}{\underline{\lfloor 15}} + \frac{598\,400\,(R\,x^3)^6}{\underline{\lfloor 21}} + \ldots\right] \ldots (28)$$

When $x = 0$, $\dfrac{d^2\,\alpha}{d\,x^2} = 0$ and, therefore, $C = 0$; when $x = L$, $\alpha = 0$; so that,

$0 = A\,\varepsilon + B\,\lambda + D\,\delta$, in which,

$$\varepsilon = \left[\frac{1}{2} + \frac{(R\,L^3)^2}{\underline{\lfloor 6}} + \frac{56\,(R\,L^3)^4}{\underline{\lfloor 12}} + \ldots\ldots\right]$$

$$\lambda = L\left[\frac{1}{6} + \frac{(R\,L^3)^2}{\underline{\lfloor 7}} + \frac{72\,(R\,L^3)^4}{\underline{\lfloor 13}} + \ldots\right]$$

and

$$\delta = L^3\left[\frac{1}{6} + \frac{20\,(R\,L^3)^2}{\underline{\lfloor 9}} + \frac{2\,200\,(R\,L^3)^4}{\underline{\lfloor 15}} + \ldots\right] \ldots\ldots\ldots\ldots (29)$$

Furthermore, when $x = L$, $\dfrac{d\,\alpha}{d\,x} = 0$, and,

$$0 = A\,\varepsilon' + B\,\lambda' + D\,\delta' \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which, the prime functions are derivatives of the corresponding functions in Equation (29) with respect to L.

When $x = 0$, $y = -a\,\alpha + n$, or, from Equations (21) and (28),

$$\frac{d\,M_3}{d\,x} = \frac{P}{b}\left(\frac{A\,a}{2} - n\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

also, from Equation (23), when $x = 0$,

$$\frac{d\,M_3}{d\,x} = \frac{E\,b\,I_z}{2}\,D \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

whence,

$$\frac{P\,a}{2\,b}\,A - \frac{E\,b\,I_z}{2}\,D - \frac{n}{b}\,P = 0 \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

Eliminating B and D from Equations (29), (30), and (33):

$$A\left[1 + \frac{E\,b^2\,I_z\,(\varepsilon\,\lambda' - \lambda\,\varepsilon')}{P\,a\,(\delta\,\lambda' - \lambda\,\delta')}\right] - \frac{2\,n}{a} = 0 \ldots\ldots\ldots\ldots (34)$$

If $n = 0$ and A is not zero, the beam is in neutral elastic equilibrium.

The development of Equations (1) to (34) indicates in a general way, the treatment of the "J-beam" and the "F-beam". Analyses have been made of the cases of the simple beam with a concentrated load at the center of the span and a theorem has been developed covering the case of the flanged beam complicated by the existence of the J-property. The treatment has also been broadened to include the unsymmetrical or channel-shaped section.

The results of analyses are shown in Figs. 5, 6, and 7. Fig. 5 illustrates the mode of failure by neutral elastic equilibrium of a flanged beam (or ideal "F-beam"; either a channel or symmetrical section). By means of Fig. 5(b), the load of neutral elastic equilibrium may be determined. Failure by neutral elastic equilibrium can occur only if the load is applied within the plane of the center of the web if a symmetrical section, or within a plane a distance, $\frac{U\,b}{2\,I_h}$, back of the center of the web in the case of the channel-shaped section.

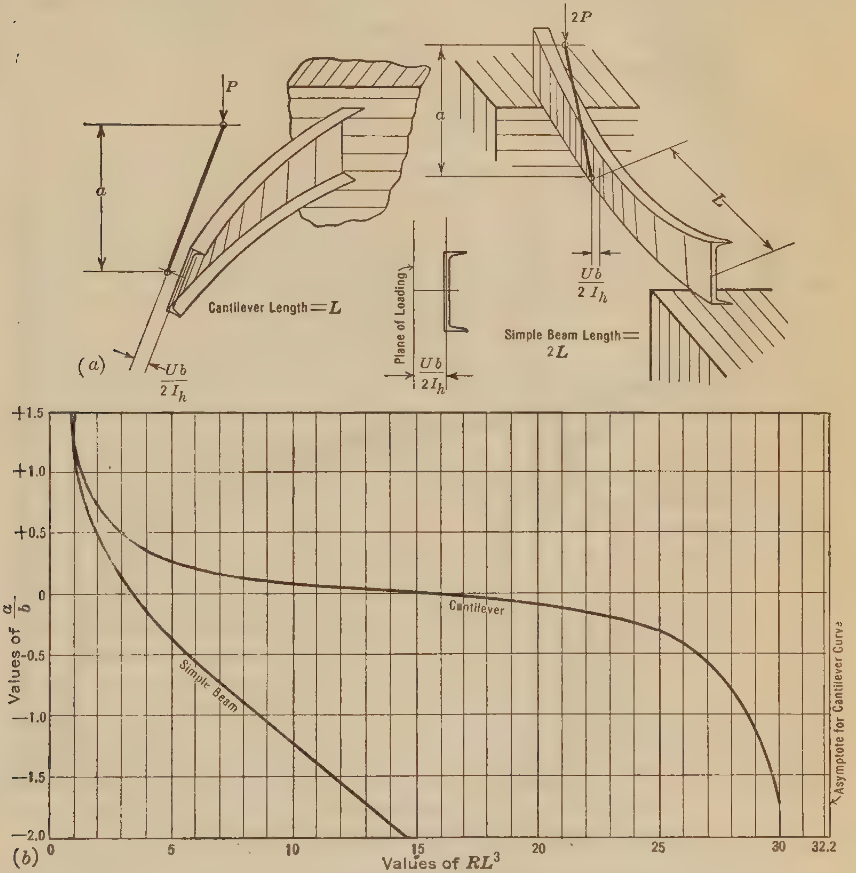

P
a
Cantilever Length = L
(a)
Ub/2I_h
Plane of Loading
Ub/2I_h
2P
a
L
Ub/2I_h
Simple Beam Length = 2L
+1.5
+1.0
+0.5
0
−0.5
−1.0
−1.5
−2.0
Values of a/b
Cantilever
Simple Beam
Asymptote for Cantilever Curve
(b)
0
5
10
15
20
25
30
32.2
Values of RL³

FIG. 5.—NEUTRAL ELASTIC EQUILIBRIUM FOR THE F-BEAM.

The factor, R, is expressed by the relation:

$$R^2 = \frac{2\,I_h\,P^2}{E^2\,b^2\,(I_h\,I_z - U^2)}\left[\frac{1}{I'} - \frac{I_z}{2\,(I_h\,I_z - U^2)} + \frac{U^2}{2\,I_h\,(I_h\,I_z - U^2)}\right] \ldots (35)$$

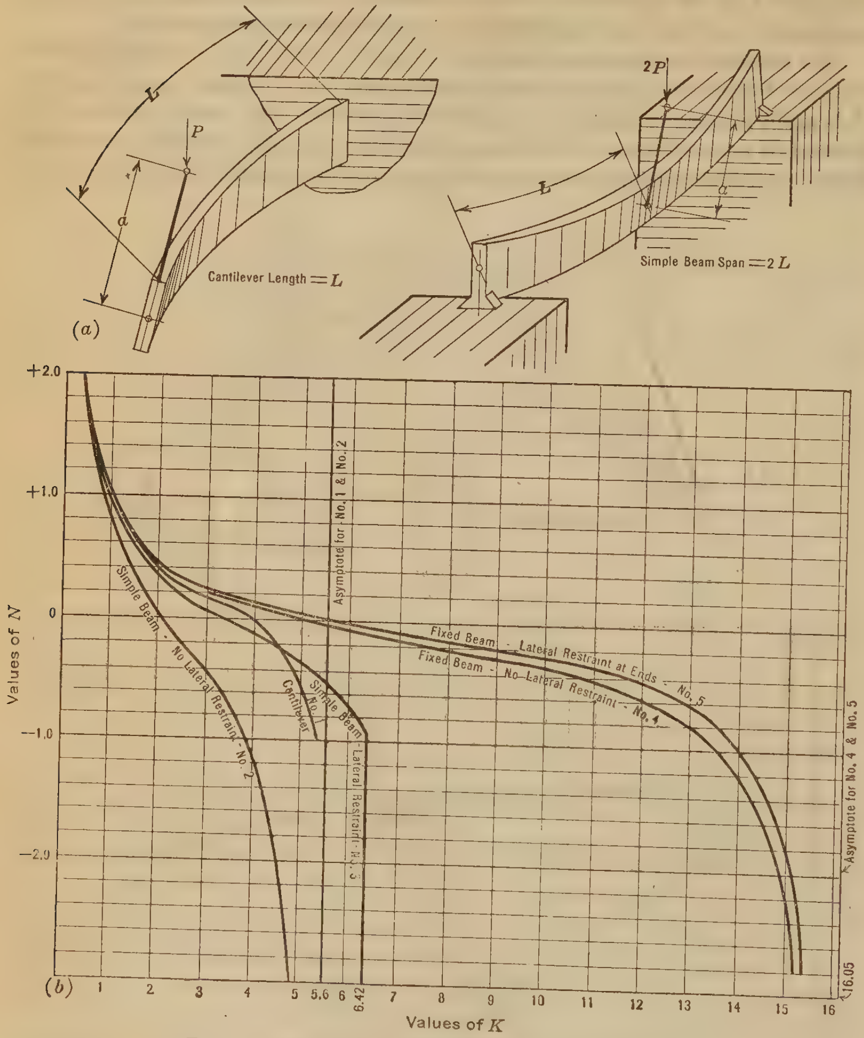


FIG. 6.—NEUTRAL ELASTIC EQUILIBRIUM FOR THE J-BEAM.

Similarly, Fig. 6 illustrates the mode of failure of the "J-beam" and Fig. 6(b) is a chart for determining the load of neutral elastic equilibrium.

Fig. 7 indicates the capacity of the standard 12-in., 31.5-lb. steel I-beam as a simple beam supported at each end and carrying a concentrated load at the middle of the span. The beam is treated as a flanged beam having torsional rigidity. Its capacity is expressed in terms of maximum fiber stress in the flange at the middle of the span when in the state of neutral elastic equilibrium.

These diagrams must not be applied to the commercial rolled sections which have a large torsional resistance property.

Table 1 gives computed properties of some standard beams and channels. The derivation of the values of the function, J, is based on results of tests of small steel beams which were made by the writer at the time of his early research in this line.

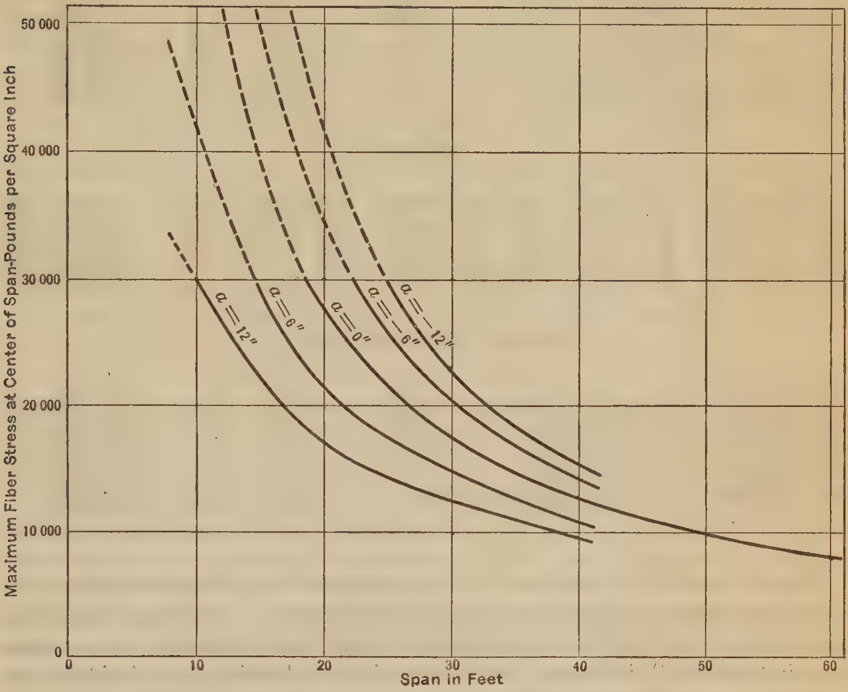


FIG. 7.—CAPACITY OF A 12-IN., 81.5-LB. STEEL I-BEAM.

Comparison will show that the usual conception of the compression flange of a beam as a member similar in principle to the column has little correspondence with the conclusions arrived at previously. As regards the "F-beam", the capacity of the beam in terms of the bending stress in the flanges varies inversely with the square of the span as would be the case if viewed as a column, but as regards the "J-beam" when $a = 0$, the capacity, in terms of the bending stress, varies inversely with the first power of the span.

The importance of the torsional resistance function is emphasized and its effect is very important in the use of the standard rolled sections. The tremendous advantage of the box-section, such as is often used in crane girders, becomes apparent and usually removes such girders from all need of serious consideration of failure by neutral elastic equilibrium.

TABLE 1.—SPECIAL PROPERTIES OF I-BEAMS, CHANNELS, AND H-COLUMNS.

Section.	I.	I_1.	I_3.	I_z.	J.	U.	b.	$\sqrt{\frac{E I_3}{E_s J}}$	$\sqrt{E I_3 E_s J}$	$\frac{U b}{2 I_h}$
I-BEAMS.										
3-in. at 5.5 lb.	2.5	0.46	0.56	....	0.040	......	2.84	5.8	2 780 000	.....
4-in. at 7.5 lb.	6.0	0.77	0.88	....	0.0692	......	3.70	5.52	4 600 000	.....
5-in. at 9.75 lb.	12.1	1.23	1.38	....	0.1017	......	4.68	5.70	7 020 000	.....
6-in. at 12.25 lb.	21.8	1.85	2.02	...	0.1428	......	5.64	5.82	9 970 000	.....
7-in. at 15.0 lb.	36.2	2.67	2.89	....	0.199	......	6.61	5.90	14 100 000	.....
8-in. at 18.0 lb.	56.9	3.78	4.05	....	0.295	......	7.63	5.75	20 350 000	.....
9-in. at 21.0 lb.	84.9	5.16	5.50	....	0.377	......	8.60	5.92	26 800 000	.....
10-in. at 25.0 lb.	122.1	6.89	7.30	...	0.507	......	9.57	5.88	35 800 000	.....
12-in. at 31.5 lb.	215.8	9.50	9.95	....	0.771	......	11.53	5.55	51 600 000	.....
15-in. at 42.0 lb.	441.8	14.62	15.1	....	1.43	......	14.38	5.02	86 500 000	.....
18-in. at 55.0 lb.	795.6	21.19	21.8	....	2.04	......	17.31	5.05	124 000 000	.....
20-in. at 65.0 lb.	1 170.0	27.86	28.6	....	3.03	......	19.21	4.75	173 500 000	.....
24-in. at 80.0 lb.	2 087.0	42.86	43.8	....	4.17	......	23.13	5.01	251 000 000	.....
CHANNELS.										
6-in. at 8.0 lb.	13.0	0.70	0.74	0.56	0.0685	1.427	5.62	5.11	4 200 000	0.616
7-in. at 9.75 lb.	21.1	0.98	1.03	0.77	0.0911	2.08	6.6	5.23	5 710 000	0.65
8-in. at 11.25 lb.	32.3	1.33	1.39	1.03	0.127	2.97	7.56	5.13	6 650 000	0.695
10-in. at 15.0 lb.	66.9	2.30	2.38	1.75	0.191	5.53	9.54	5.50	12 570 000	0.787
12-in. at 20.5 lb.	128.1	3.91	4.03	2.89	0.351	9.78	11.44	5.26	22 200 000	0.87
15-in. at 33.0 lb.	312.6	8.23	8.45	5.80	0.905	21.5	14.3	4.75	51 500 000	0.985
H-COLUMNS (BETHLEHEM SECTION).										
6-in. at 20.0 lb.	38.7	13.0	19.6	....	0.241	......	5.65	14.0	40 500 000	.....
8-in. at 32.0 lb.	105.0	35.8	54.0	....	0.567	......	7.47	15.2	103 000 000	.....
10-in. at 49.5 lb.	263.0	89.1	134.0	....	1.39	......	9.35	15.3	265 000 000	.....
12-in. at 65.5 lb.	499.0	168.0	254.0	....	2.31	......	11.1	16.3	452 000 000	.....
14-in. at 84.0 lb.	885.0	294.0	442.0	....	3.48	......	13.1	17.6	731 000 000	.....
14-in. at 107.5 lb.	1 166.0	389.0	584.0	....	6.93	......	13.3	14.2	1 188 000 000	.....
14-in. at 123.5 lb.	1 364.0	454.0	680.0	....	10.3	......	13.4	12.6	1 560 000 000	.....
14-in. at 139.0 lb.	1 568.0	520.0	780.0	....	14.2	......	13.5	11.5	1 950 000 000	.....

The old method of treatment is too conservative and uneconomical when used in the design of simple and wide spreading frames. Engineers are finding such frames more and more desirable, especially for the outdoor type of structure in which all intricacy and elaboration of bracing are decidedly undesirable from the standpoint of maintenance, and it is hoped that the sketchy treatment offered will lead the way to further study of the subject along these lines.

DISCUSSION

Jacob Feld,* Assoc. M. Am. Soc. C. E. (by letter).—The author opens up a subject which can fill many volumes, and which is the very basis of structural design, and then discusses in detail one phase of the subject, namely, the true action of two types of structural members acting as beams.

The great majority of present-day designs are based on formulas which are known to be purely approximate. The unknown as well as the omitted factors in determining maximum stress, are provided for by the use of a unit stress which is admittedly much lower than that at which members may fail. This method, of course, cannot be classed as the scientific procedure and has as its virtue only one fact, that a very small percentage of structures so designed have been known to fail.

It seems to the writer that three questions must be completely answered before a scientific design procedure can be expected: (1) What type of stress causes failure; (2) what is the actual distribution of stress through a section; and (3) what re-distribution of stresses occurs in a section when one fiber is stressed beyond its elastic limit? The writer makes no attempt to discuss these points.

Of course, it is fairly simple to determine all the possible loading conditions and, if time is available, to investigate stresses resulting from every possible loading; but the maximum stress which exists under any one condition of loads and the true excess of strength before failure occurs cannot be determined so easily. If failure has occurred, as in a test specimen, the type of stress causing failure often can be determined from an inspection of the fracture.

In general, the actual stresses, both theoretical and experimental, are greater than the apparent stresses. The true or equivalent stresses are a refinement over the apparent stresses caused by the addition of combined normal and shear stresses as opposed to normal stresses only.

Experimental determination of stresses in various members acting in bending has shown greater stresses than those calculated by the ordinary empirical methods. However, the measured values agree closely with those computed by the ordinary elastic theory if due allowance is made for lateral strain (Poisson's ratio).†

In dealing with beams under bending and torsion the author has introduced a subject in which the ordinary analysis gives results that do not agree with experimental data. It is only necessary to point out the great variation between analytical and experimental results for the effective polar moment of inertias as tabulated in a report of tests on the torsional strength of steel I-sections.‡

The effect of torsion on beams in ordinary building construction, although never considered in the average design, is marked by unsightly results in many structures. This is especially apparent in spandrel beams in concrete structures which very often show cracks on the exterior face at approximately the quarter points of the span. Similar torsional strains are visible on steel buildings, in which case the brick or stone facing shows open joints.

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† "The Strength of I-Beams and Girders," by Herbert F. Moore and W. M. Wilson, *Bulletin No. 86*, Eng. Experiment Station, Univ. of Illinois, 1916.

‡ *Bulletin 4*, Univ. of Toronto, 1924.

However, in the type of frames for which the author develops his theory, the effect of torsion due to eccentricity of loading can be included in the analysis. It must be remembered, however, that experimental results also show a variation of shear stress across the depth of the web, the shearing stress being much greater in that part of the web nearest the loaded flange than near the unloaded flange. Therefore, if a beam first has an unequal unit shear stress in the web area, even with axial load, the condition of neutral elastic equilibrium used by the author as a criterion for the strength of a beam, must take into account the combination of shear and tension or compression at the junction of the web and flange areas near the loaded flange.*

With reference to the author's discussion of the proper use of channel members as beams, it is proper that several contributions to the subject be enumerated. In a thesis presented at the University of Zurich, in 1920, H. Schwyzer proved mathematically that, in order to produce bending without twisting, a channel section must be loaded by forces parallel to the web and a distance, $\frac{3\,b}{6 + \frac{F_1}{F_2}}$, from the back of the channel. The factor, b, is the width of the flange, F_1 is the area of the web, and F_2 is the area of one flange.

A detailed method for the calculation of torsional stresses has been developed by Dr. A. Eggenschwyler.† Other contributions to the subject are those of Maillart‡ and Zimmerman.§ These men have checked their analyses against the experimental work of Bach who found among other things that the longitudinal stress at the connection of the web and flange of channel under load is 36% greater on the compression side, and 53% greater on the tension side, than the values given by the bending formula.

If the author suggests that structures be designed for the maximum allowable stress without a factor of safety, he must give methods of analysis which will determine the maximum existing stress under any given loads. He is to be commended for his addition to the ultimate goal of all design analysis.

F. B. SEELY,‖ W. J. PUTNAM,‖ and W. L. SCHWALBE,‖ ESQS. (by letter).—The author points out that when a channel is used as a beam without lateral restraint the stresses set up in it will not be in accordance with the simple flexure formula, $f = \frac{M\,h}{2\,I_h}$, unless the "position of the loading system

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* "Observed Stress Distribution in Web of I-Beam," by O. H. Basquin, *Engineering News-Record*, June 3, 1926.

† *Eisenbau*, 1921; *Bauingenieur*, 1922.

‡ *Schweizerische Bauzeitung*, April 30, 1921.

§ *Bauingenieur*, April 30, 1921.

‖ Professors, Dept. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

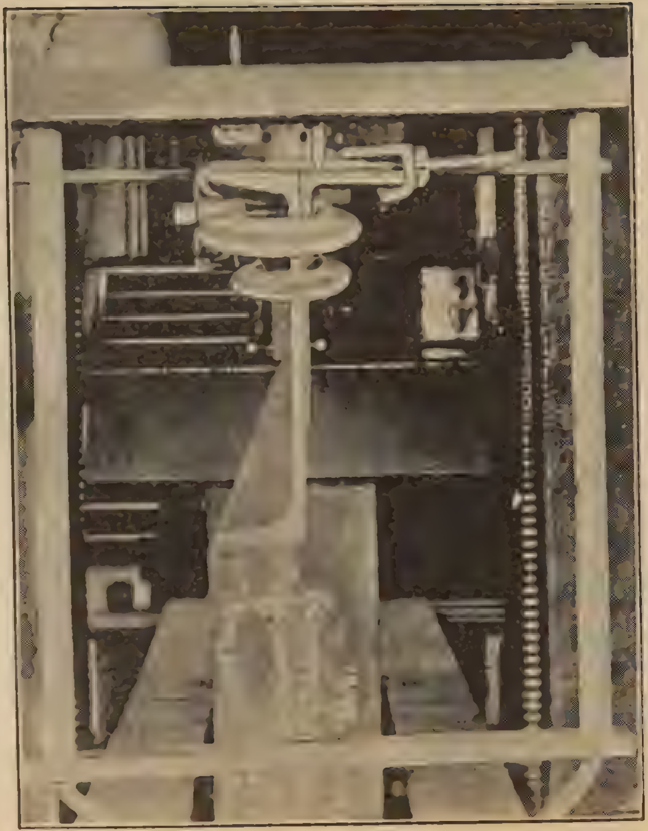

FIG. 8.— VIEW OF CHANNEL WITH LOAD APPLIED THROUGH SHEAR-CENTER AT END OF CANTILEVER.

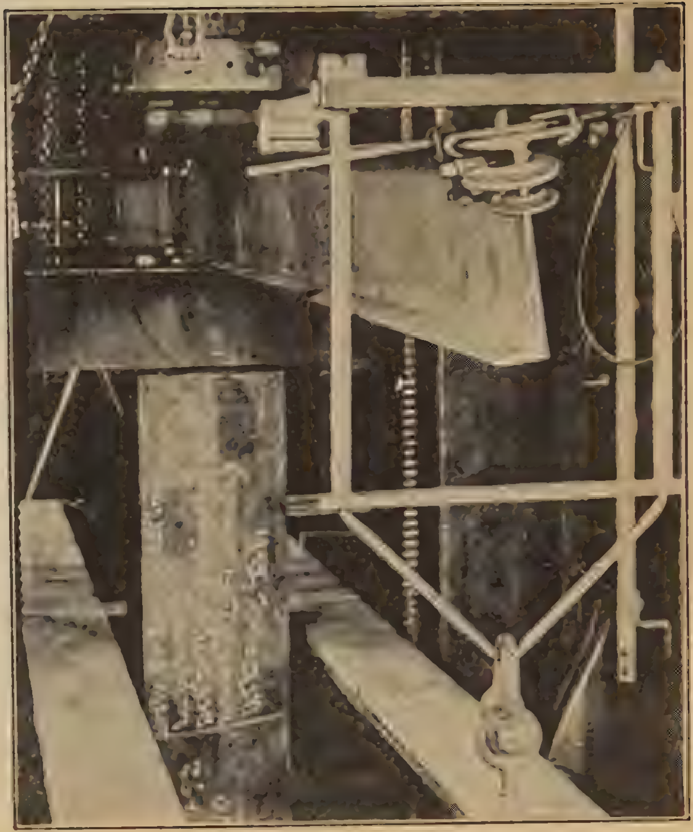

FIG. 9.—VIEW OF CHANNEL WITH LOAD APPLIED AT POINT ON FLANGE DIRECTLY ABOVE CENTER OF GRAVITY OF END SECTION WHEN SECTION IS NOT TWISTED.

is that of a plane which lies parallel to the web and a distance back from the center of the web equal to $\frac{U\,b}{2\,I_h}$."

It has been very widely assumed, however, that the loads should be applied through the center of gravity of the sections in order to cause a channel to bend without twisting and to produce stresses in accordance with the simple flexure formula.

It may be worth while, therefore, to present experimental evidence of the fact that a channel must be loaded back of the back of the channel (not through the center of gravity of the cross-sections) in order to cause bending, without twisting, as is assumed in the simple theory of beams.

The writers have made bending tests* of a number of rolled steel channels, varying in depth from 4 to 15 in., in which the positions of the loads that caused bending in accordance with the flexure formula were determined. The main purpose of the tests, however, was to determine the additional longitudinal stress in a channel beam when loaded so that twisting accompanies bending. This additional longitudinal stress, for some channels, when the load acts through the center of gravity of the section may be more than one-half the maximum stress caused by the vertical bending moment.

The name, "shear-center", has been used to denote the point on the axis of symmetry of the channel through which the plane of loads must pass to produce simple bending (without twisting), and its location and significance seem to have been brought first to the attention of engineers by R. Maillart† in 1921, and by A. Eggenschwyler‡ and A. Ostenfeld§ in 1924. In 1925, K. Hubert‖ obtained experimental results on a rolled steel channel 10.3 in. (26.14 cm.) deep, tested as a simple beam on a span of 20.8 ft. (5.8 m.), and subjected to a concentrated load of 5 500 lb. (2 500 kg.) at mid-span; the position of the "shear-center," as found from the test results, was in substantial agreement with the calculated position.

In order to determine the "shear-centers" of the channels tested at the University of Illinois, each channel was tested as a horizontal cantilever beam with a vertical load consisting of a suspended weight applied at the free end. Some of the smaller channels were embedded at one end in a large cube of concrete, and this cube was held on the weighing table of a testing machine by applying a load to the top face of the cube by means of the movable head of the testing machine. The large channels were tested as double cantilevers. In Fig. 8, a 15-in., 33.9-lb. channel is shown with a load of 5 600 lb. applied through the "shear-center" at the end of a 10-ft. cantilever. In Fig. 9, the same load is applied at a point on the flange directly above the center of gravity of the end section when the section is not twisted. The beam was balanced on a knife-edge on the weighing table of a testing machine and equal weights were applied to the two free ends. The central

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* In the Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

† *Schweizerische Bauzeitung*, Vol. 77, pp. 195–198 (1921).

‡ *Loc. cit.*, Vol. 83, pp. 259–262 (1924).

§ "Teknisk Elästicitetslære," Fourth Edition (1924).

‖ *Der Bauingenieur*, Vol. 6, p. 458 (1925).

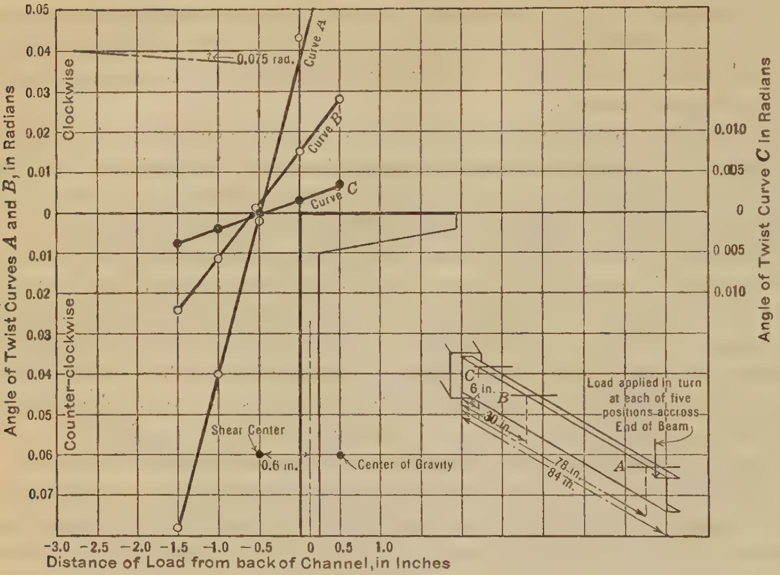

FIG. 10.—LOCATION OF SHEAR-CENTER FOR 6-INCH, 8.2-POUND CHANNEL.

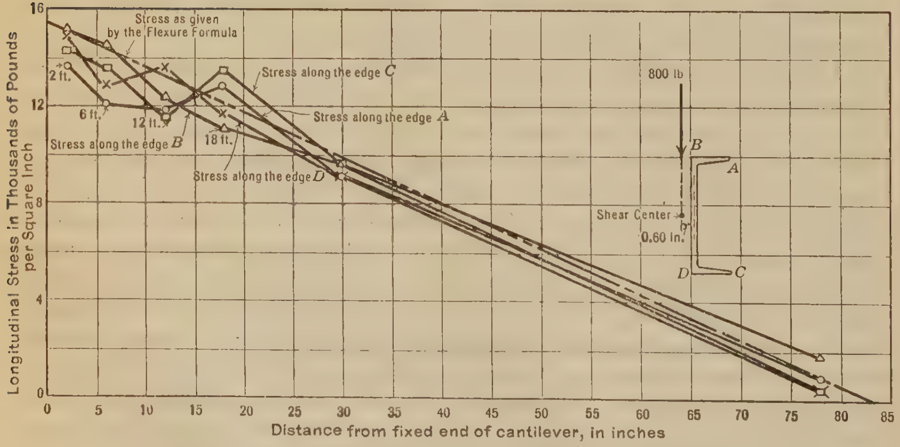

FIG. 11.—LONGITUDINAL STRESS IN 6-INCH, 8.2-POUND CHANNEL WITH LOAD OF 800 POUNDS APPLIED THROUGH THE SHEAR-CENTER AT END OF CANTILEVER HAVING A LENGTH OF 84 INCHES.

section of the channel was held fixed in position by a light load applied through the movable head of the testing machine to the top flange, and by small horizontal jacks on each side of the top and bottom flanges. In addition, the outstanding flanges at the central section were supported by a vertical stiffener extending between the flanges.

The angle of twist was measured at six or seven sections along the length of the beam for each of several lateral positions of the load at the end. The load was applied at two or three points on each side of the back. An angle was clamped to the back at the free end of the beam, so that the load could be applied back of the back of the channel. (See Fig. 8.)

The angle of twist was measured by means of a level bar used on a 20-in. gauge length. For this purpose bars slightly more than 20 in. long were clamped to the top flange of the channel at several sections along the length of the beam, and the change in inclination of these bars was measured on a 20-in. gauge length; the change in elevation of one end of the gauge length with respect to the other was measured to 0.001 in. by adjusting a micrometer screw at one end of the level bar. The bubble in the level bar was a 20-sec. bubble, which was amply sensitive.

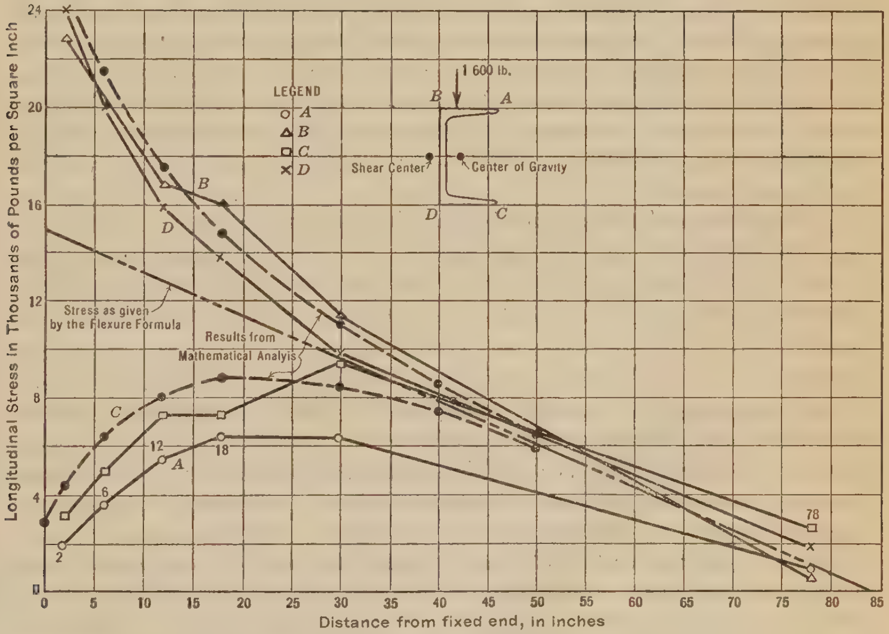

FIG. 12.—EXPERIMENTAL VALUES OF LONGITUDINAL STRESS IN A 6-INCH, 15-POUND SHIP CHANNEL.

The values of the angles of twist at three sections for five different lateral positions of the load on a 6-in., 8.2-lb. channel are given in Fig. 10. (One point on Curve *A*, at + 0.5 in. = 0.0735 radians, has been omitted.) The diagram shows clearly that this channel beam twists, as it bends, when the load is applied at any lateral position, except that at the distance of

0.60 + in., approximately, from the center of the web. The calculated value for the distance of this "shear-center" from the center of the web, according to the expression, $\frac{U\,b}{2\,I_h}$, is 0.618 in. A satisfactory agreement between experimental and calculated results was obtained for the other channels tested.

In order to determine the longitudinal stresses in the channel beam, the strains along each of the four edges were measured with a 2-in. strain-gauge at several sections. A value of 30 000 000 lb. per sq. in. was used for the modulus of elasticity in converting strains to stresses. Readings of the gauge could be repeated within a maximum error of ± 1 000 lb. per sq. in.

Fig. 11 shows the experimental values of the longitudinal stresses in the 6-in., 8.2-lb. channel beam when the load of 800 lb. is applied through the "shear-center" as determined from Fig. 10. It is evident that the stresses are in substantial agreement with those given by the simple flexure formula.

Fig. 12 shows the experimental values of the longitudinal stresses in a 6-in., 15-lb. ship channel when the vertical load of 1 600 lb. is applied at the point on the flange that is directly above the center of gravity of the section before the beam is stressed. As in Fig. 11 the beam is a cantilever with a length of 84 in. After the load is applied and the beam twists, the load no longer acts through the center of gravity of the section. It is clear that the twisting of the beam causes the stresses along two edges near the fixed section to be greatly in excess of those given by the simple flexure formula, and those along the other two edges to be much less than those found from the flexure formula.

If, however, a channel is used where it has lateral support so that it is restrained from twisting as it bends, this additional longitudinal stress does not develop; furthermore, due to the small torsional stiffness of channels, a relatively small amount of lateral restraint is sufficient to prevent the twisting.

Norman Litchfield,* Esq. (by letter).—This paper is of unusual interest in that it represents the results of many years of study in connection with the development of structures used largely for the support of electric wires on many of the major electrifications of steam railroads in the United States. Formerly, designers of such structures followed somewhat conservatively along the lines of standard bridge construction. The repetition of such structures every 250 or 300 ft. along the railroad right of way gives a vista of extreme heaviness. Furthermore, the ordinary bridge type of structure with its latticed members and re-entrant corners is difficult to maintain, especially on roads that operate a combination of electric and steam service, a condition which has obtained on a great many lines. These pockets collect cinders and corrosion is very rapid.

The structures that Mr. Richmond has aided in developing are light in appearance, but a careful study of the designs, the mathematics of which are developed in the paper, will indicate that they are thoroughly sound. Moreover, many years' experience has shown that they are entirely satisfactory from all standpoints, including safety, durability, and economical maintenance.

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* With Gibbs & Hill, Cons. Engrs., New York, N. Y.

The subject which the author presents so well is one of increasing interest to railroad companies as congestion on many lines is gradually forcing more extended consideration of electrification. There has been an attempt to write into some specifications for structures of this character certain provisions based on the accepted practice of bridge design. Should it be necessary to follow these in exact conformity, a considerable hardship would be put upon the railroads that are using electric traction, because they would be forced to erect uneconomical structures which are also difficult to maintain. The writer believes Mr. Richmond's paper will be received with considerable interest by all structural engineers who face the problem of the support of electric wires.

E. R. HILL,* M. AM. SOC. C. E. (by letter).—Structures designed in accordance with the principles set forth in this paper have been used in connection with railroad electrification and power transmission for many years. They are relatively light, and, although flexible, possess ample strength under service and emergency conditions. It is noteworthy that of thousands of structures designed in this way and erected over the main-line tracks of railroads, there has been no case of failure or evidence of undue strain.

For the extensive electrification of main-line railroads the design of structures for supporting the trolley wires and transmission and signal circuits assumes a very important place. Such structures should involve minimum first cost and maintenance expense consistent with the satisfactory performance of the service required, and because of their extensive use they should be neat and attractive in appearance, avoiding the suggestion of a mass of steel over or along the tracks. A clear view of signals should be maintained. There is also a great duplication of structures of the same design, or similarity of type, and engineers can well afford to devote much detail study to an accurate analysis of the loads which must be carried, both normally and in emergencies. Where such determinations are made Mr. Richmond has found it practicable and entirely permissible to abandon certain of the empirical limitations that are generally imposed in the design of structures and develop methods of calculation and design especially applicable to these structures.

O. G. JULIAN,† M. AM. SOC. C. E., and GEORGE R. RICH,‡ ASSOC. M. AM. SOC. C. E. (by letter).—The well known and widely used reduction formula,

$$\frac{M}{S} \overline{<} \frac{f}{1 + \left(\frac{L}{b}\right)^2 \frac{1}{k}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

merely considers the unsupported length of the compression flange as a column. The same buckling resistance is assigned to all beams having a given width of flange, irrespective of the "shape factor" (that is, the relation that the stiffness

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[*] Cons. Engr. (Gibbs & Hill), New York, N. Y.

[†] Catenary Engr., Jackson & Moreland, Hoboken, N. J.

[‡] Engr., Hydr. Div., Stone & Webster Eng. Corporation, Boston, Mass.

of the member about the axis parallel to bending bears to its stiffness about the axis normal to bending), or the factor of torsional rigidity of the section. The author's rational analysis clearly indicates that the flexural rigidity about both axes and also the torsional rigidity are vital factors in resisting this type of failure.

The critical buckling load for beams of narrow rectangular cross-section and also for I-beams in which the flexural rigidity about the axis normal to the plane of loading is great in comparison with that about the axis parallel to the plane of loading, has been determined analytically for a wide range of loads and terminal conditions by Professor S. Timoshenko* by considering the changes in potential energy of the elastic system. The writers have attempted to extend Professor Timoshenko's analysis so as to include H-beams, built-up girders, and trusses, loaded either parallel or normal to the major axis, and to develop practical working formulas for such members which would not penalize them unduly as appears to be done by the formulas in common use. The values so obtained for critical loads check the author's values fairly closely.

According to Professor Timoshenko's theory† as understood by the writers, it is required to evaluate the following changes in potential energy, or work done, at the instant buckling and twisting occurs:

V_1, due to bending about the axis normal to the plane of loading;
V_2, due to the bending of each flange or chord about the axis parallel to the plane of loading;
V_3, due to twisting of the section; and,
V_4, due to the movement of the loads in their plane of application.

In so far as buckling and twisting are concerned, as long as $V_1 + V_2 + V_3 > V_4$, the member is in "stable elastic equilibrium" and as load is added it will finally fail when the $\frac{M}{S}$ stress exceeds the yield point of the material. When $V_1 + V_2 + V_3 = V_4$, "neutral elastic equilibrium" exists, and as further increments of load are added, the potential energy of the elastic system is decreased and the member may fail by buckling or twisting.

In that which follows, a cantilever beam subjected to a concentrated load in the plane of one principal axis has been selected for study. However, the method appears to be general and applicable in the analysis of members subjected to various loads and types of end restraint.

Using the author's nomenclature, except as noted, and if W = work done, f = fiber stress in general, and λ = change in length,

$$W = \frac{1}{2} \int f \, d \, A \, \lambda \quad \text{.......................(37)}$$

which, provided the elastic limit of the material has not been exceeded, becomes (prior to buckling):

$$W = \frac{1}{2} \int_0^L \frac{M^2 \, d \, x}{E \, I} \quad \text{.......................(38)}$$

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* "Beams Without Lateral Support," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 1247.

† *Loc. cit.*, p. 1257.

or, for beams of constant cross-section and homogeneous material,

$$W = \frac{P^2}{2\,E\,I} \int_0^L x^2\,d\,x \text{ ....} (39)$$

As buckling and twisting occur the load, P, may be resolved into components normal and parallel to the original axis of loading, as indicated in Fig. 13 (*c*). Then,

$$W = \frac{P^2}{2\,E\,I} \int_0^L (1 - \alpha^2)\,x^2\,d\,x \text{ ....} (40)$$

and by subtracting Equation (39) from Equation (40),

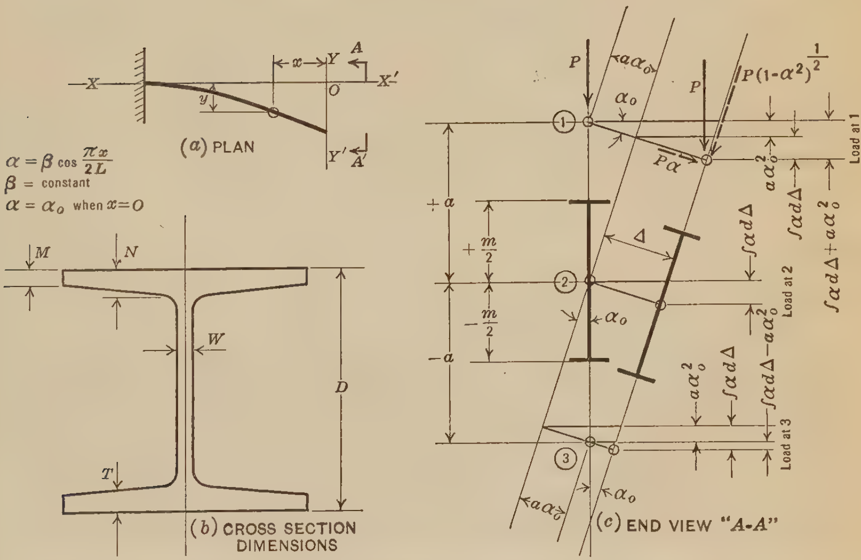
$$V_1 = -\frac{P^2}{2\,E\,I} \int_0^L x^2\,\alpha^2\,d\,x \text{ ....} (41)$$

Similarly,

$$V_2 = \frac{P^2}{2\,E\,I'} \int_0^L x^2\,\alpha^2\,d\,x \text{ ....} (42)$$

or,

$$V_2 = \frac{E\,I'}{2} \int_0^L \left(\frac{d^2\,y}{d\,x^2}\right)^2 d\,x \text{ ....} (43)$$



A
—X
x
Y
X′
O
y
(a) PLAN
Y′
A′
$\alpha = \beta \cos \frac{\pi x}{2L}$
β = constant
$\alpha = \alpha_o$ when $x=0$
M
N
W
D
T
(b) CROSS SECTION DIMENSIONS
P
$a\alpha_o$
α_o
P
$P(1-\alpha^2)^{\frac{1}{2}}$
①
$P\alpha$
$a\alpha_o^2$
$\int \alpha\,d\Delta$
$\int \alpha\,d\Delta + a\alpha_o^2$
Load at 1
+a
$+\frac{m}{2}$
Δ
②
$-\frac{m}{2}$
α_o
−a
Load at 2
$\int \alpha\,d\Delta - a\alpha_o^2$
$-a\alpha_o^2$
$\int \alpha\,d\Delta$
Load at 3
③
$a\alpha_o$
α_o
(c) END VIEW "A-A"

FIG. 13.—THE EFFECT OF BENDING AND TWISTING.

Considering the work done on the flanges separately, it is possible to replace y by $y + \frac{m\,\alpha}{2}$ for the upper flange and by $y - \frac{m\,\alpha}{2}$ for the lower flange. Then,

$$V_2 = \frac{P^2}{2\,E\,I'} \int_0^L x^2\,\alpha^2\,d\,x + \frac{E\,I'}{8}\,m^2 \int_0^L \left(\frac{d^2\,\alpha}{d\,x^2}\right)^2 d\,x \text{ ......} (44)$$

The work done in twisting a member may be expressed by,

$$V_3 = \frac{1}{2} \int Q\,d\,\alpha \text{ ....} (45)$$

but, in general, and as given by the author,

$$\alpha = \frac{Q\,x}{E_s\,J} \quad \text{.........................} (46)$$

Then,

$$V_3 = \frac{E_s\,J}{2}\int_0^L \left(\frac{d\,\alpha}{d\,x}\right)^2 d\,x \quad \text{..................} (47)$$

Referring to Fig. 13 (c), it may be seen that,

$$V_4 = P\,a\,\alpha_0^2 + P\int \alpha\,d\,\varDelta \quad \text{..................} (48)$$

but,

$$d\,\varDelta = \frac{P}{E\,I'}\,\alpha\,x^2\,d\,x \quad \text{......................} (49)$$

Then,

$$V_4 = P\,a\,\alpha_0^2 + \frac{P^2}{E\,I'}\int_0^L x^2\,\alpha^2\,d\,x \quad \text{................} (50)$$

For the system to be in "neutral elastic equilibrium",

$$-\frac{P^2}{2\,E\,I}\int_0^L x^2\,\alpha^2\,d\,x + \frac{P^2}{2\,E\,I'}\int_0^L x^2\,\alpha^2\,d\,x + \frac{E\,I'}{8}\,m^2\int_0^L \left(\frac{d^2\,\alpha}{d\,x^2}\right)^2 d\,x$$
$$+\frac{E_s\,J}{2}\int_0^L\left(\frac{d\,\alpha}{d\,x}\right)^2 d\,x - \frac{P^2}{E\,I'}\int_0^L x^2\,\alpha^2\,d\,x - P\,a\,\alpha_0^2 = 0 \ldots (51)$$

The value of α equals 0 when x equals L, and is a maximum when x equals 0; therefore, if β is a constant, it may be said that,

$$\alpha = \beta\cos\frac{\pi\,x}{2\,L}{}^{*} \quad \text{........................} (52)$$

Then,

$$P^2\left(\frac{1}{2\,E\,I} + \frac{1}{2\,E\,I'}\right)\left(\frac{L^3}{6} - \frac{L^3}{\pi^2}\right) + P\,a - \frac{E\,I'\,m^2\,\pi^4}{256\,L^3} - \frac{E_s\,J\,\pi^2}{16\,L} = 0 \,..\, (53)$$

Nine curves are drawn in Fig. 14 showing "Compression Stresses for Various Unsupported Lengths of Compression Flange". A comparison is made of various formulas and various beams. Curves 6, 7, and 8 are plotted from formulas which limit the unit compressive stress to working values with a factor of safety of approximately two; the remaining curves give the ultimate values of the unit compression at which buckling failure is incipient. In obtaining data for Curves 5 and 9, the test load rested directly on the top flange of the beam corresponding to a value of $a = +\frac{m}{2}$. In Curves 1, 2, 3, 4, 6, and 7, the value of a was assumed equal to 0; that is, the load was applied at the center of the web. Equation (53) was used to calculate Curves 2 and 4, the values of J being obtained from the formula proposed by Mr. William B. Campbell:†

$$J = 0.4\,D\,W^3 + 0.1\,(N + M)^3\,(T - W) \quad \text{............} (54)$$

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* "Beams Without Lateral Support," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p 1248.

† *Engineering News-Record*, October 11, 1928, p. 542.

in which, *D*, *W*, *N*, *M*, and *T* are as shown in Fig. 13 (*b*). The numerical values of *J* for the curves in Fig. 14 are as follows:

Curve No.	Value of *J*.
1	2.31
2	2.53
3	0.295
4	0.322

There is substantial agreement between values of *J* obtained by Equation (54) and those given in Table 1. Both the author and Mr. Campbell predicated their values on tests of **I**-beams. It is doubtful whether the results should be extended to include **H**-beams in which the comparatively heavy flanges carry a greater proportion of the torsional stress. Experimental data on the torsional resistance of **H**-sections seem to be entirely lacking.

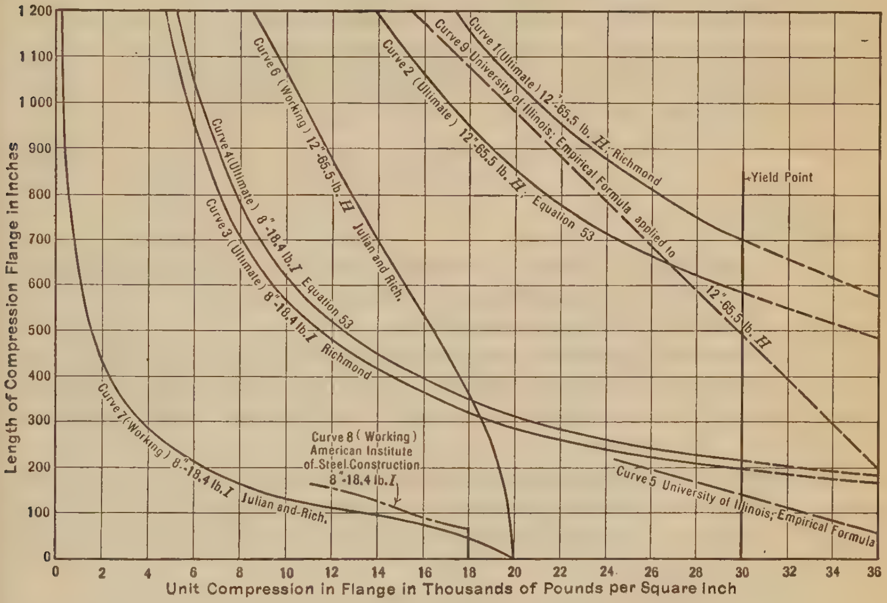

FIG. 14.—COMPRESSIVE STRESSES FOR VARIOUS UNSUPPORTED LENGTHS OF COMPRESSION FLANGE.

As successive increments of load are applied to a beam in bending about the axis normal to the web, the plane form of flexure remains stable up to a certain point; that is, if an accidental lateral force were to disturb the beam, on removal of the force the beam would return to the original plane. As the load is further increased, a sudden lateral deflection occurs, accompanied by torsion. In other words, on application of the last load increment the potential energy of the elastic system decreased, because the last load increment caused a relatively great increase of the work done by the additional dropping of the load as measured by Equation (50). The beam seeks relief by rotation of the section toward a more favorable position as regards lateral components of the loading.

As successive increments of load are applied to the same beam, but in bending about the axis of the web, the plane form of bending remains stable provided the component parts of the beam act as a unit. Failure will finally occur when the yield point of the material is exceeded, because the section is already in the position which affords maximum relief from buckling and twisting and could not possibly rotate toward a more stable position as regards lateral components of the loading.

When tubes and similar sections are subjected to bending no reduction formula for unit compression in the extreme fiber is necessary, because buckling will not occur unless torsional rotation can move the section to a more favorable position.

The author's analysis indicates that an efficient working formula for limiting stress intensity in the compression flange should evaluate the "shape factor" of the section. When members are loaded about the axis normal to the web it should allow higher intensities in **H**-beams than in **I**-beams, and when loaded about the axis of the web, it should require little or no reduction in intensity of stress in the compression fibers, provided the member will act as a unit. The following formula seems to satisfy this criterion:

$$\frac{M}{S} \overline{<} \frac{f}{1 + \left(\frac{L}{c'\,r_p}\right)^2 \left(\frac{r_n}{r_p}\right)^2 \frac{c}{k}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (55)$$

in which,

f = maximum allowable extreme fiber stress due to direct bending plus that due to buckling and twisting.

L = length of the compression flange.

r_p = radius of gyration about the axis parallel to the plane of loading.

r_n = radius of gyration about the axis normal to the plane of loading.

c, c', and k are constants, dependent in value on the "shape factor" of the member.

The writers wished to arrive at a formula which would limit f to 20 000 lb. per sq. in., such that, although it would be applicable to beams and trusses of almost any section, it would be directly applicable and best adaptable to **H**-sections. The formula derived was:

$$\frac{M}{S} \overline{<} \frac{20\,000}{\dfrac{1 + \left(\frac{L}{4\,r_p}\right)^2 \left(\frac{r_n}{r_p}\right)^2 \frac{1}{3}}{8\,000}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (56)$$

For **H**-sections having "shape factors" similar to that of a 12-in., 65.5-lb. **H**-beam and loaded parallel to the web, Equation (56) becomes:

$$\frac{M}{S} \overline{<} \frac{20\,000}{1 + \left(\frac{L}{b}\right)^2 \frac{1}{8\,000}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (57)$$

in which, b = breadth of the flange. If the loading is normal to the web, it becomes practically, $\frac{M}{S} \overline{<} 20\,000$.

For I-beams having "shape factors" similar to that of an 8-in., 18-lb. I-beam and loaded parallel to the web, Equation (56) becomes:

$$\frac{M}{S} \overline{<} \frac{20\,000}{1 + \left(\frac{L}{b}\right)^2 \frac{1}{1\,100}} \qquad (58)$$

In Fig. 15, the denominator of Equation (56) has been plotted against L for various sections. The one exception to this is for the curve marked "American Institute of Steel Construction", for which the abscissas are determined by the quantity, $1 + \dfrac{\left(\frac{L}{b}\right)^2}{2\,000}$.

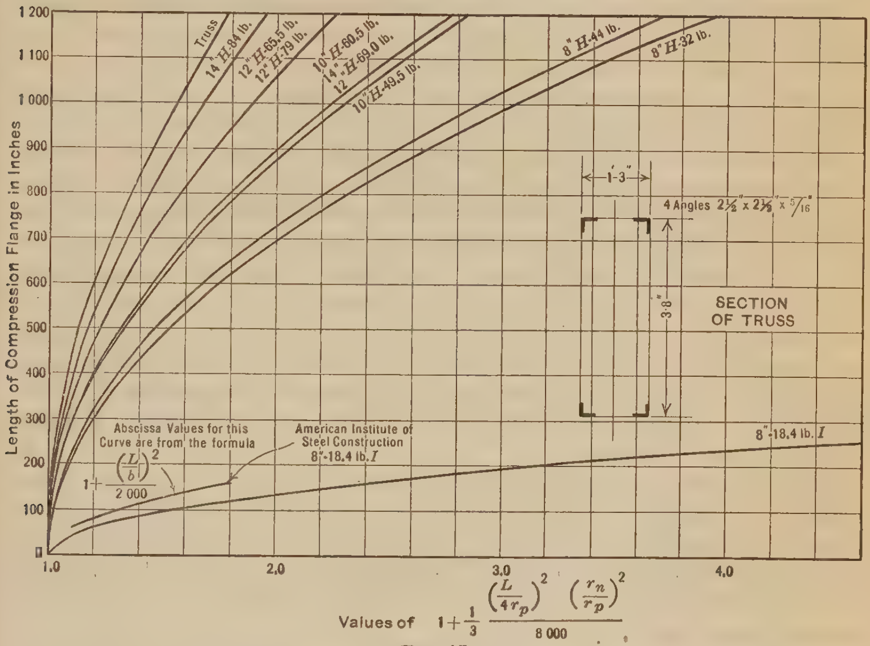

FIG. 15.

The writers were able to find very little test data on side buckling of the compression flange and torsional resistance of such sections as H-columns. Curve 5 of Fig. 14 has been plotted from tests made at the University of Illinois* of I-beams loaded on the top flange. The empirical formula thus derived was:

$$\frac{M}{S} = 40\,000 - \frac{60\,L}{r_p} \qquad (59)$$

So far as the writers know, similar tests have not been made on H-sections. It is felt that such tests would confirm the author's analytical findings.

The writers wish to express their appreciation of this splendid and timely paper. Structural engineers engaged on railway electrification would owe the author an additional debt of gratitude if he would publish his method of evaluating the resistance of beam sections when subjected to a torsional load, such as that encountered due to "broken wire" loadings.

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* "Tests of I-Beams in Flexure," by Prof. Herbert F. Moore, *Bulletin No. 68*, Univ. of Illinois Eng. Experiment Station.

JOHN I. PARCEL,[*] M. AM. SOC. C. E. (by letter).—This excellent paper is a timely contribution to the theory of structures. It is very true, as the author notes, that the buckling of beams has received far less attention than the buckling of columns. There are doubtless good reasons why the latter should have taken a more prominent place in the literature of the subject, but it is none the less true that the "elastic equilibrium" of beams has not been given the place its importance deserves, at least in American technical literature.

The author states that, so far as he knows, this and two other brief papers constitute the only analytical treatment of the problem. Without any intention of minimizing the importance of Mr. Richmond's independent and original analysis, it seems pertinent to call attention to the very considerable literature on the subject that has been in existence for some time.

The first treatment of the problem appears to be due to Professor L. Prandtl, of the University of Göttingen. His original paper[†] covers very fully the basic theory of the "tipping" of long, thin beams in flexure and includes a detailed treatment of what the author designates as "*J*-beams".

The first study of flanged beams, I-sections in particular, appears to be due to Professor S. Timoshenko.[‡]. His treatment was on essentially broader lines than that devoted to "*F*-beams" in the paper, covering quite fully the case referred to by the author as the "flanged beam complicated by the existence of the *J*-property". In later papers,[§] Professor Timoshenko has developed his theory in some detail for both simple and cantilever beams of I-section under a variety of loadings.

The interesting fact that a channel section, in order to develop a pure bending when loaded in a plane parallel to the web, must have the load placed a certain distance back from the web (see Fig. 5), appears to have been first observed by Professor C. von Bach. The analysis of the flexure and torsion of channels has been the subject of many papers appearing in the German technical press since about 1920.[‖]

Although the author presents some very interesting and useful tables and graphs, the prime object of his paper seems to be the presentation of a fundamental method for the rational analysis of the problem of the stability of a narrow beam subjected to flexure. In view of this fact, the presentation of a somewhat different method of approach leading to the same results may be of some interest.

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[*] Cons. Engr. (Sverdrup & Parcel), St. Louis, Mo.

[†] "Kipperscheinungen," (Munich Dissertation), 1899. A very clear and excellent résumé of Prandtl's theory will be found in the treatise by H. Lorenz, "Technische Elastizitätslehre" (1913), pp. 353–365.

[‡] First presented (in Russian) in a *Bulletin* of the Technical Inst. of St. Petersburg, 1906.

[§] "Einige Stabilitäts Probleme der Elastizitätstheorie," *Zeitschrift f. Math. u. Phys.*, 1910, Vol. 58, pp. 337–385; and "Sur la Stabilité des Systèmes Elastiques," *Annales des Ponts et Chaussées*, Vol. III, pp. 496–566; Vol. IV, pp. 73–132; and Vol. V, pp. 372–412.

[‖] See paper by Professor L. Föppl, *Der Bauingenieur*, Heft 12, 1925, p. 455, *et seq.*, in which a considerable bibliography is given.

The writer assumes Equations (4) to (7), inclusive, as the basic formulas, except that the right side of Equations (4), (6), and (7) is assumed to be negative. The author notes that in a flanged beam fixed at one end and carrying a concentrated load at the other, the principal torsional resistance is furnished by a lateral bending moment in the flanges, $\pm M_3$. Assuming that all distortions are small enough to justify the application of the theory of elasticity, it is possible to write, with sufficient exactness:

$$\eta = \frac{b}{2} \alpha \qquad \text{(60)*}$$

in which, η = the linear displacement of the center line of the flanges in a direction normal to the plane of the web, due to the twisting effect alone; and from the ordinary theory of bending,

$$\pm M_3 = E I_f \frac{d^2 \eta}{d x^2} \qquad \text{(61)}$$

if I_f = moment of inertia of one flange in the plane of bending. Furthermore,

$$\pm \frac{d M_3}{d x} = E I_f \frac{d^3 \eta}{d x^3} = \frac{b}{2} E I_f \frac{d^3 \alpha}{d x^3} = V \qquad \text{(62)}$$

The pair of oppositely directed shears in the top and bottom flanges, due to the lateral bending moment, M_3, effectively constitute a couple:

$$V b = b \frac{d M_3}{d x} = \frac{b^2}{2} E I_f \frac{d^3 \alpha}{d x^3} = K \frac{d^3 \alpha}{d x^3} \qquad \text{(63)}$$

if $K = \frac{b^2}{2} E I_f$.

If, in addition to the resistance to twist furnished by the lateral bending of the flanges, the beam also develops a purely torsional resistance, it follows that, from Equations (4), (5), and (63):

$$Q = P \left(y - x \frac{d y}{d x} \right) = K \frac{d^3 \alpha}{d x^3} - C \frac{d \alpha}{d x} \qquad \text{(64)}$$

if $C = E_s J$. Since the torque increases as α decreases, a negative sign is placed before the last term. Differentiating Equation (64) and taking account of Equation (6):

$$K \frac{d^4 \alpha}{d x^4} - C \frac{d^2 \alpha}{d x^2} - \frac{P^2 x^2}{E} \left(\frac{1}{I_1} - \frac{1}{I} \right) \alpha = 0 \qquad \text{(65)}$$

which is the differential equation of elastic equilibrium of the cantilever subjected to a load, P, at the free end and slightly bent and twisted.

If the beam is incapable of developing a purely torsional resistance, C is equal to zero and, since, $I_1 = 2 I_z$; $I = 2 I_h$, and $I_f = I_z$ (approximately):

$$\frac{d^4 \alpha}{d x^4} - R^2 x^2 \alpha = 0 \qquad \text{(66)}$$

in which,

$$R^2 = \frac{P^2}{E^2 b^2 I_z} \left[\frac{1}{I_z} - \frac{1}{I_h} \right]$$

which is Equation (27) for the "F-beam".

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* This is essentially the same as the author's equations, top of p. 855.

If, on the other hand, the beam is of such a form that the contribution of the lateral bending of the flange to the twisting resistance is negligible, K is equal to zero, and the differential equation of elastic equilibrium for the J-beam is:*

$$\frac{d^2 \alpha}{d x^2} + N^2 x^2 \alpha = 0 \ldots\ldots\ldots\ldots (67)$$

if,

$$N^2 = \frac{P^2}{E E_s J}\left(\frac{1}{I_1} - \frac{1}{I}\right).$$

A primitive of this differential equation may be found in terms of Bessel's functions,† or it may be integrated by means of a simple power series in the same manner as the author has integrated Equations (10) and (27). As an example of method, it may be interesting to sketch the process (for this rather simple case) in more detail than is given in the paper.

Assume that α can be expressed in a converging infinite series in ascending powers of x, thus: $\alpha = A_0 + A_1 x + A_2 x^2 + A_3 x^3 + \ldots\ldots$ Then,

$$\frac{d^2 \alpha}{d x^2} = 2 A_2 + 2 \times 3 A_3 x + 3 \times 4 A_4 x^2 \ldots.$$

Equation (67) then gives,

$$2 A_2 + 2 \times 3 A_3 x + 3 \times 4 A_4 x^2 + \ldots + N^2 A_0 x^2 + N^2 A_1 x^3 + N^2 A_2 x^4 + \ldots = 0 \ldots\ldots\ldots\ldots (68)$$

If Equation (68) is to hold for all values of x, the method of undetermined coefficients requires that: $A_2 = A_3 = 0$; $A_4 = \frac{-N^2 A_0}{3 \times 4}$; $A_5 = \frac{-N^2 A_1}{4 \times 5}$; $A_6 = A_7 = 0$; and, in general, $A_n = \frac{-N^2 A_{n-4}}{n(n-1)}$.

All coefficients are thus expressed in terms of the two arbitrary factors, A_0 and A_1, which are the constants of integration for the second-order differential equation, and then:

$$\alpha = A_0\left(1 - \frac{N^2 x^4}{3 \times 4} + \frac{N^4 x^8}{3 \times 4 \times 7 \times 8} - \ldots\right) + A_1 x \left(1 - \frac{N^2 x^4}{4 \times 5} + \frac{N^4 x^8}{4 \times 5 \times 8 \times 9} - \ldots\right) \ldots\ldots\ldots\ldots (69)$$

If it is desired to obtain the expression for the horizontal displacement, y, this may readily be done by means of Equation (7), thus:

$$E_s J \frac{d \alpha}{d x} = - P\left(y - x \frac{d y}{d x}\right) = - P x^2 \frac{d}{d x}\left(\frac{y}{x}\right)$$

from which,

$$\frac{1}{x^2}\frac{d \alpha}{d x} = - \frac{P}{C}\frac{d}{d x}\left(\frac{y}{x}\right), \text{ and } - y = \frac{C}{P} x \int \frac{1}{x^2}\frac{d \alpha}{d x} d x + B x \ldots (70)$$

in which, B is a new integration constant.

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* The preceding analysis is essentially the same as that presented by A. and L. Föppl, "Drang und Zwang," First Edition, Vol. II, p. 350 *et seq.*

† See A. E. H. Love, "Mathematical Theory of Elasticity," 1920 Edition, pp. 425-427.

If the value of α from Equation (69) is substituted in Equation (70);

$$-y = B\,x + A_0'\left(-x^3 + \frac{N^2\,x^7}{2 \times 6 \times 7} - \frac{N^4\,x^{11}}{2 \times 7 \times 8 \times 10 \times 11} + \dots\right)$$

$$+ A_1'\left(-1 - \frac{N^2\,x^4}{3 \times 4} + \frac{N^4\,x^8}{4 \times 5 \times 7 \times 8} - \dots\right) \dots\dots\dots\dots (71)$$

in which, $A_0' = \frac{1}{6}\,\frac{C\,N^2}{P}\,A_1$ and $A_1' = \frac{C}{P}\,A_1$. Equation (71) is substantially identical with Equation (11).

From the relations arising in the evaluation of the constants, the author arrives at a value for the lowest critical load of,

$$P_k = 4.014\,\sqrt{\frac{E\,I_3\,E_s\,J}{L^4}}$$

Professor Prandtl has determined a value of,

$$P_k = 4.0126\,\sqrt{\frac{E\,I_1\,E_s\,J}{L^4}}$$

Noting that $I_3 = \dfrac{I_1}{1 - \dfrac{I_1}{I}}$, and that $\dfrac{I_1}{I}$ is always negligibly small in any case in which the "tipping" action of the beam is of any practical importance, the correspondence between the two results is practically exact.

The author states that he has extended his analysis to cover the case of flanged beams having an appreciable true torsional resistance, although this analysis is not included in the paper. It may be noted that the differential Equation (65) which applies to this case, may be integrated in precisely the same manner as the author has in Equations (10) and (27). The solution has been completely described by Professor Timoshenko,* who has also provided tables for use in the numerical application of his results. It should be observed that these tables are only intended for the case in which the load is assumed to be applied at the center of gravity of the cross-section ($n = 0$; $a = 0$ in the author's notation). Fig. 5 (*b*) is the only solution the writer has seen which takes into account the effect of the application of the load at other points than the centroid.

It is interesting to compare the critical load derived by Professor Timoshenko for a given case with that obtained from the author's curve. In the example under consideration,† the cantilever is a plate-girder section, with approximately the following properties: Span, 16 ft., 5 in.; depth, 1 ft., 7½ in.; $I = 670$; $I_1 = 6.8$; and the section is made up of 2⅜ by 2⅜ by $\frac{5}{16}$-in. angles and 19½ by $\frac{5}{16}$-in. web.

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* "Einige Stabilitäts Probleme der Elastizitätstheorie," *Zeitschrift f. Math. u. Phys.*, 1910, Vol. 58, pp. 337–385; and "Sur la Stabilité des Systèmes Elastiques," *Annales des Ponts et Chaussées*, Vol. III, pp. 496-566; Vol. IV, pp. 73-132; and Vol. V, pp. 372-412.

† "Einige Stabilitäts Probleme der Elastizitätstheorie," *Zeitschrift f. Math. u. Phys.*, 1910, Vol. 58, p. 372.

Then, from Fig. 5 (*b*),

$$a = 0, R\,L^3 = 15 = \text{(approximately)}\ \frac{P\,L^3}{E\,b\,\frac{I_1}{2}}$$

and,

$$P = \frac{15 \times 29\,000\,000 \times 19.5 \times 3.4}{197 \times 197 \times 197} = 3\,790 \text{ lb.}$$

The corresponding value obtained by applying Professor Timoshenko's tables is 3 840 lb. This indicates that the effect of the torsional resistance in a girder of such dimensions as the one considered is very slight compared to the resistance furnished by the lateral bending of the flanges, and since this is fairly evident from general considerations, the close correspondence of the two values constitutes a good check on the analysis.

S. TIMOSHENKO,[*] ESQ. (by letter).—The author raises an important point in showing the necessity for investigations into the elastic stability of structures. This becomes especially important in light structures containing slender members, such as are used in railroad electrification. In such cases, a failure of the structure may occur, due to instability rather than to high stresses. Increasing the strength of the steel used does not increase safety in such cases, because the critical load for a slender member depends on the modulus of elasticity of the material, and not on its strength.

As an important example of elastic instability, the author discusses the problem of sidewise buckling of beams loaded in the plane of maximum rigidity. Two extreme conditions are considered: (1) Beams in which torsional resistance plays an important part in providing lateral stability; and (2) beams in which torsional resistance is due to bending of the flanges only. The first condition is fulfilled to a great extent in the case of beams of narrow rectangular cross-section. This problem has been discussed by L. Prandtl[†] and A. G. M. Michell.[‡] Both arrived at the same equation for a cantilever as that given by the author, namely:

$$P = \frac{4.014\sqrt{B\,C}}{L^2} \quad\text{..........................}\quad (72)$$

in which, B is the flexural rigidity of the beam in a lateral direction; and C is the torsional rigidity of the beam. In the case of an I-beam, the problem becomes more complicated because not only torsional rigidity of the beam, but also the resistance of the flanges to bending, must be considered. The writer[§] has shown that instead of Equation (72), the following equation must be used:

$$P_c = \frac{K\sqrt{B\,C}}{L^2} \quad\text{..........................}\quad (73)$$

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[*] Prof. of Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

[†] "Kipperscheinungen," Dissertation, München, 1899.

[‡] *Philosophical Magazine*, Vol. 48, 1899.

[§] See *Bulletin*, Polytechnical Inst. at St. Petersburg, 1905–06; see, also, *Zeitschrift für Mathematik und Physik*, 1910, and *Annales des Ponts et Chaussées*, 1913.

in which, K is a numerical factor depending on the proportions of the beam. Some values of this factor for various values of the ratio, $\frac{L^2}{a^2} = \frac{4\,C\,L^2}{B\,d^2}$, are as given in Table 2.

TABLE 2.—NUMERICAL VALUES OF THE FACTOR, K, IN EQUATION (73).

$\frac{L^2}{a^2}$.	K.	$\frac{L^2}{a^2}$.	K.
0.1	44.3	10	7.58
1	15.7	12	7.20
2	12.2	14	6.96
3	10.7	16	6.73
4	9.76	24	6.19
6	8.69	32	5.87
8	8.03	40	5.64

The factor, d, refers to the over-all depth of the beam. For greater values of $\frac{L^2}{a^2}$, the factor, K, can be calculated from the following approximate equation:

$$K = \frac{4.01}{(L - a)^2} \quad \text{.........................} (74)$$

It may be seen that, as the ratio, $\frac{L^2}{a^2}$, increases, the factor, K, approaches 4.01, and Equation (73) coincides with Equation (72). This means that for long I-beams the bending of the flanges becomes of secondary importance. The author also gives without derivation some numerical results for a 12-in., 31.5-lb., steel I-beam supported at the ends and loaded at the middle. These results are in satisfactory agreement with results given by the writer.* For example, when $L = 20$ ft. and $a = 0$ (that is, the point of application of the load coincides with the centroid of the cross-section), the writer obtains† $f = 27\,000$ lb. per sq. in. This agrees with the ordinate in Fig. 7. It should be noted that the author determines the torsional rigidity by tests of small steel beams, although this quantity can be calculated with sufficient accuracy by using one of two approximate formulas proposed by the writer.‡

In conclusion, it should be noted that the question of the stability of a given configuration is connected with the investigation of the energy of the system in this configuration. The configuration is stable provided the corresponding energy is a minimum. On this basis of considering the energy of the system an approximate method for calculating the critical values of loads has been developed in a paper by the writer published elsewhere in this volume.

The method proved very handy for approximate solutions of stability problems, and in the case of sidewise buckling of beams it is simpler than the method used by the author, which is based on the investigation of the corresponding differential equation of equilibrium.

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* *Transactions*, Am. Soc. C. E., Vol. 87 (1924), p. 1247.

† *Loc. cit.*, Table 5, p. 1255.

‡ *Loc. cit.*, p. 1249.

Furthermore, by using the energy method, problems can be discussed in which a member is subjected simultaneously to the "primary loading", and to a "secondary loading", and it can be proved that, in general, a member subject to deformation by the primary loading will not be so well adapted to resist the effects of secondary loading as if the first system of loading did not exist. Applying the method to struts submitted to axial compression (the primary loading), and lateral loading (the secondary loading), it can be proved that when all the lateral forces are in the same direction the "load-resisting function of the member as regards the secondary system is approximately proportional to 1 minus the ratio of the primary load or system to a similar load or system which would result in a condition of neutral elastic equilibrium."

The problem of determining the center of twist in bending of non-symmetrical profiles, which is raised by the author (Fig. 5) in discussing bending of channel members, is also of practical importance, and has been extensively discussed in recent European literature.[*] The case of a cantilever in which the cross-section is a sector of a circle, or is bounded by two concentric arcs and two radii, is discussed in detail by M. Seegar and K. Pearson.[†] The case in which the section is an isosceles triangle is treated by the writer.[‡] The problem of determining the center of twist in the general case of a non-symmetrical profile can also be solved with sufficient accuracy, provided the thickness of the material is so small that the distribution of the shearing stress over it can be taken as uniform. This problem has been discussed in recent engineering literature by several authors.[§]

H. S. Richmond,[‖] Esq. (by letter).—One of the discussers, Mr. Feld, brings forward three questions having to do with stress distribution which he states the writer makes no attempt to discuss. The treatment of this subject in detail lies outside the scope of the paper. As indicated by Professor Timoshenko, it is assumed in treating problems in elastic stability that failure is to occur, not through over-stress, but by excessive distortion below the yield point. The analyses are based upon relations prevailing below the elastic limit.

Recognition, however, is given in the paper to the weakening effect of a load similar to the crippling load, but less in value; but the designer must work out the stress relations in the particular design, making proper application of the theorems.

The writer's values for the torsional resistance functions of I and H-sections are only tentative. In theoretical studies, it should be recognized that this function not only varies from the polar moment of inertia, but that for these types of section, it actually has no relation thereto.

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[*] See, A. E. H. Love, "Theory of Elasticity," IV Edition, 1927, p. 340.

[†] *Proceedings*, Royal Soc. (Lond.), (Series A), Vol. 96 (1920), p. 211.

[‡] *Proceedings*, Mathematical Soc. (Lond.), (Series 2), Vol. 20, 1922, p. 398.

[§] See R. Maillart, *Schweizerische Bauzeitung*, Vol. 77, p. 197; Vol. 79, p. 254; Vol. 83, pp. 111 and 176; C. Weber, *Zeitschrift für angewandte Mathematik und Mechanik*, Vol. 4, 1924, p. 334; and A. Eggenschwyler, *Proceedings*, Second International Cong. of Mechanics, Zurich, 1926.

[‖] With Gibbs & Hill, Cons. Engrs., New York, N. Y.

The discussion by Messrs. Scely, Putnam, and Schwalbe is exceedingly interesting in illustration of the experimental determination of the "shear-center" in channels. The writer has not seen any discussion of this principle in the standard works on structural mechanics. Students have been free to arrive at all sorts of erroneous conclusions as to the action of channels as flexural members.

Messrs. Julian and Rich have made an interesting analysis and have prepared a working formula which, for the practical range which lies safely within the limit of the curve for the crippling load and for low slenderness ratios, meets an arbitrarily assigned working stress. It is to be noted that their formula, Equation (53), is based upon the condition, $a = 0$, and that their working formula assumes this; therefore, care should be taken to see whether that condition prevails. In many problems, this is an important consideration.

It would be possible to derive a working formula for any given class and use of beam; the writer, however, would prefer not to crystallize the theorems into working formulas at this time, but would express the hope that the principles brought into consideration might receive further study and in due course be given recognition in the drafting of codes of design, especially where light and open-work types of construction are concerned.

The writer is obligated to Mr. Parcel and other commentators for citations of analyses by previous investigators, showing points of correspondence with his own work.

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Paper No. 1746

PUMPED-STORAGE HYDRO-ELECTRIC PLANTS*

By William W. K. Freeman,† Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. John R. Freeman, H. Alden Foster, Oren Reed, Paul L. Heslop, A. H. Markwart, and William W. K. Freeman.

Synopsis

The Rocky River hydro-electric development, recently completed near New Milford, Conn., is the first modern plant in America to pump water into a reservoir with off-peak power, in order to produce on-peak power later. In connection with this project, a search of engineering literature was made for data on similar plants built elsewhere. The number of such plants was discovered to be forty-two. Ten other plants have been proposed in Europe (including one 600 000-kw. installation) and thirty-six in America. The rapid increase in size of units and total installations and the number of plants built in Europe in the past few years, when considered with the building of the first large plant in the United States, make it appear that America is on the threshold of a new phase in hydro-electric development.

The principle of operation of such plants is therefore explained, with a discussion of their economic justification; a summary is given to point out the extremes in physical features and the advanced stage of development of this type of hydro-electric development; and brief descriptions of the most interesting pumped-storage plants are included, particularly of those which are adaptations of existing developments. Their fundamental cycle of operation is discussed in the Appendix.

I.—General Description

Growth and Size.—An increasing number of American hydraulic engineers are becoming interested in the type of hydro-electric development in which water is pumped into a reservoir with off-peak energy in order to produce on-peak energy later. Few of them, however, know that this has been done for nearly half a century; that forty-two plants have been built to date (Fig. 1); that twenty-one of them have been built within the past five years; and that their evolution has progressed to the point where 34 000-h.p. pumps are being installed and a total pumping capacity of 250 000 kw. has been seriously proposed (see Tables 1 and 2). Ten more plants have been proposed in Europe, where practically all this development has taken place. In the United States, however, the first modern plant of this type has recently been completed by the Connecticut Light and Power Company on Rocky River, near Milford, Conn., although thirty-six more plants have been proposed, among them being one to pump 5 000 sec-ft. against a 90-ft. head over a divide.

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* Published in November, 1928, *Proceedings*.

† Asst. Engr., New England Power Constr. Co., Boston, Mass.

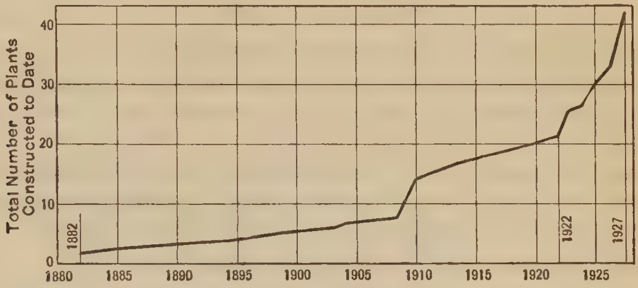

FIG. 1.—HYDRO-ELECTRIC PLANTS WITH STORAGE RESERVOIRS FILLED BY PUMPING.

Those who are in close touch with the situation regard the Rocky River development as the first of a large number of such plants in this country. The inevitable human inertia due to a conservative attitude toward the transplanting of a European idea to American soil has now been overcome and with the building of the first large plant in the United States the development should be rapid. It is timely therefore to explain the principles of operation and to show the wide range of conditions for which these plants have been built, as well as to stress their advanced stage of evolution.

"Hydraulic accumulator" is a name that has been given such plants, ill-advisedly, in the writer's opinion, because an hydraulic accumulator is a system used principally in rolling mills for supplying jacks with water under high pressure. A preferable term is, therefore, the more graphic "pumped-storage hydro-electric plant".

PRINCIPLE OF OPERATION

The principle on which these plants operate is that of building a storage reservoir where the natural conditions are favorable for the economic storage and utilization of water even if the natural inflow into the reservoir is negligible, provided there exists, a short distance away and at a lower level, a supply from which water can be pumped into the reservoir. This pumping is done with low-cost off-peak energy generated at steam plants or run-of-stream plants, in order later to obtain high-value peak energy from the storage reservoir. The process of using power to pump water into a reservoir and drawing it out again to obtain power would be uneconomical except for the fact that electric energy must be produced at the time the consumer requires it. The marginal cost of off-peak energy from steam turbo-generators which would be idle or partly loaded if it were not for the pumping load of a pumped-storage plant, is quite low; and the marginal cost of off-peak energy from water which would otherwise be wasted over the dams of run-of-stream plants (called "surplus hydro-electric energy") is practically zero. These conditions tend to make it economical to use this equipment (which would otherwise be wholly or partly idle, but on which nevertheless the fixed charges would be continuing), to pump water into a reservoir to produce power later when required, the arrangement resembling an electric storage battery in its operation.

TYPES OF PLANTS

Storage Type.—There are three types of plants, the storage development and two variations of it—the pondage and the part-head types. Rocky River is a plant of the storage type (see Fig. 2), in which two principles of operation are used. By the first, water is drawn from the reservoir to generate on-peak power in this plant only, and is replaced by an equal quantity within the same day or week, pumped into the reservoir with off-peak power. In this case, there is no net depletion of the reservoir, and the plant is said to operate on the pondage (that is, short-time storage) principle.

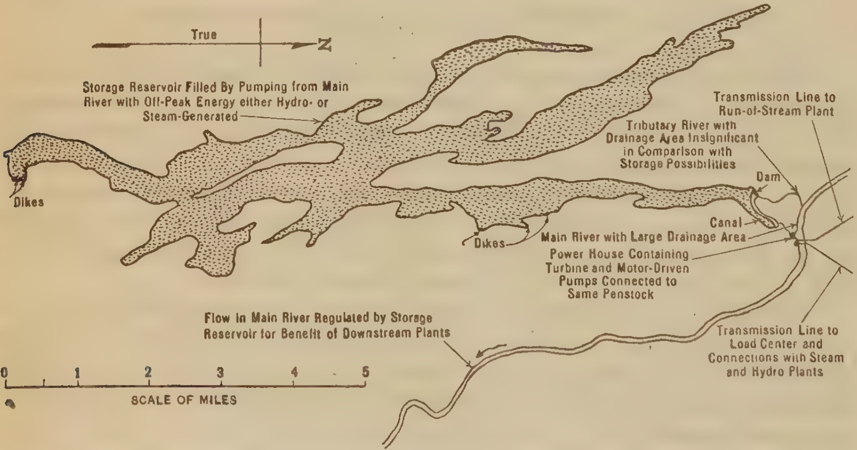
True
N
Storage Reservoir Filled By Pumping from Main River with Off-Peak Energy either Hydro- or Steam-Generated
Dikes
Transmission Line to Run-of-Stream Plant
Tributary River with Drainage Area Insignificant in Comparison with Storage Possibilities
Dam
Canal
Dikes
Main River with Large Drainage Area
Power House Containing Turbine and Motor-Driven Pumps Connected to Same Penstock
Flow in Main River Regulated by Storage Reservoir for Benefit of Downstream Plants
Transmission Line to Load Center and Connections with Steam and Hydro Plants
0
1
2
3
4
5
SCALE OF MILES

FIG. 2.—TYPICAL PUMPED-STORAGE HYDRO-ELECTRIC PLANT, ROCKY RIVER, NEAR NEW MILFORD, CONN.

The second is the flow-regulation principle in which water is drawn from the reservoir not only to produce power at this plant, but also to increase the low-water flow in the main river for the benefit of down-stream plants. This operation differs from that common to all storage reservoirs only in the fact that the reservoir is filled by pumping because the natural inflow is too small.

Pondage Type.—Nearly four times as many pondage plants as storage plants have been built. In this type only short-time storage is economical and the feature of releasing water to regulate the flow of the river below the reservoir, as described for the storage type, is entirely absent. Some have pumps of enormous size; as, for instance, the Hengstey plant now under construction, which has 34 000-h.p. pumps. Among the developments of medium size are some reservoirs formed by dikes on hillsides, while a few of them are so small that they are made of reinforced concrete.

A number of the plants of this type have two reservoirs, one forming the forebay of the plant and the other the tail-race. In this case the only additional water used, once the lower reservoir is filled, is that required to replace leakage and evaporation. This type of plant extends the possibilities of hydro-electric development to locations where it has never been considered, because practically no natural flow of water is necessary. It requires only suitable natural or artificial pools at different elevations, with the proper hydraulic works.

Part-Head Type.—As illustrated in Fig. 3, the third or part-head type pumps water against a certain head to make it available through a higher head.

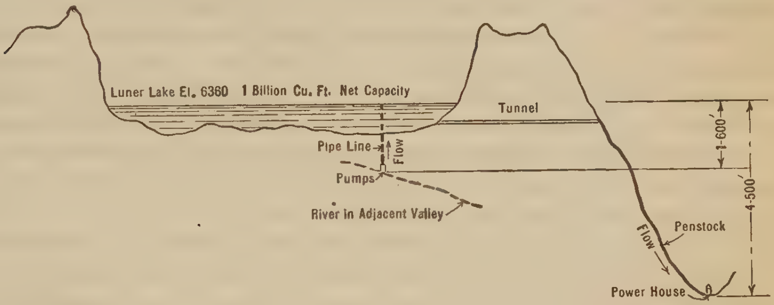

FIG. 3.—TYPICAL PART-HEAD PLANT AT LÜNER LAKE, AUSTRIA.

A common form of part-head development is one in which water is pumped into the penstock from a lake, or other source of supply, and from there it flows up into the main reservoir. The only American plant built prior to Rocky River on which data have been found, is one of this type. In addition to the Mill Creek plant, pumping is utilized at the Thousand Springs, Lake Waha, and Bear Lake developments in Idaho, but these are not true pumped-storage plants, because at Thousand Springs the pumping is done continuously and there is no storage of water during off-peak periods; and at the other two the water is pumped out of a reservoir which is filled by natural inflow, instead of into a reservoir deficient in natural inflow.

ECONOMIC JUSTIFICATION

Allusion has been made to the economics of pumped-storage plants. To discuss this question let it be assumed that, due to the growth of its load, a certain company requires additional firm capacity which can be supplied by the enlargement of its steam plant or by the addition of a pumped-storage plant to its hydro-electric system, in which there are some run-of-stream plants. The net value of the hydro-electric plant, therefore, is determined in the following manner. Hydraulic computations for a certain installed capacity show:

(1) The increase in the firm capacity of the system due to the building of the pumped-storage plant;
(2) The increase in the primary energy;
(3) The decrease in the secondary energy produced by the hydro-electric system due to storing water during periods of higher flow; and
(4) The secondary energy used for pumping.

It is considered properly conservative to assume that all pumping both for pondage and for storage will be done by steam-generated off-peak energy (although a part of it may be done by water otherwise wasted over the dams of run-of-stream plants). It is likewise conservative to omit the saving due to the plant's production of secondary energy except from the natural inflow into the reservoir, as discussed in the Appendix.

The value of the hydro-electric plant is the sum of its "capacity value" plus its "energy value", and is measured by the fixed charges and the production costs, respectively, at the alternative steam plant. The meaning of the term, "capacity value", is best explained by an illustration. Thus, if a pumped-storage plant adds 75 000 kw. of firm capacity to the system, and the alternative addition to the existing steam plant would consist of two 40 000-kw. units at a cost of $9 600 000, the fixed charges at 11% would be $1 060 000, or $13 per kw. per year. The capacity value of the 75 000-kw. hydro-electric plant at $13 per kw. per year is, therefore, $975 000 per year.

Various methods are used to calculate the energy value of a hydro-electric plant. By one method the value of the energy is taken as the production cost of the same energy at the alternative steam plant. For a pumped-storage plant the energy value is found by subtracting from the value of the primary energy, the sum of the values of the decrease in secondary energy due to storing water plus the secondary energy used for pumping.

Against the capacity value plus the energy value of the hydro-electric plant, are set the fixed charges on the investment in the pumped-storage plant plus its production costs. The difference represents the annual saving through the building of the pumped-storage plant instead of an addition to the steam plant, or *vice versa*. In addition to these items there are also frequently others, such as fixed charges on the unused capacities of the alternative installations before they are fully loaded; investment and operating costs of transmission lines; relative reliability; and the addition of an independent source of supply. Some of these may easily be evaluated; while others cannot be evaluated, but should be considered.

As may readily be seen for any specific case, the economic justification of the pumped-storage plant lies in its capacity value even more than in the difference in production cost between off-peak and on-peak steam-generated energy.

General Economic Considerations

The writer hesitates to try to formulate general statements as to what conditions make plants of the three different types economical because of the infinite variations there can be in topography, market, competing power, and surplus power for pumping, all of which influence vitally the economy of any project.

In general, however, it may be said that the conditions that make ordinary hydro-electric plants feasible make pumped-storage plants feasible; for example, low investment cost per kilowatt of firm capacity; high cost of competing fuels; large and concentrated system loads, etc. Although the pumped-storage plant labors under the disadvantages of the added cost of the pumping equipment and operation, it has certain compensating advantages over ordinary storage plants, some of which may be enumerated as follows:

1.—It is comparatively independent of rainfall and drainage area—although, of course, a high average annual rainfall on a large drainage area means less pumping.

2.—In more settled areas the land and other damages are apt to cost less for a given reservoir capacity because the plant is on a tributary instead of on the main river.

3.—Since, for a given reservoir capacity, the drainage area is smaller, the provision for floods is less expensive, if not absent.

4.—The plant that pumps from a large river, or from a pool, is valuable as a reserve plant because water released in an emergency can be replaced by pumping.

5.—Finally, among the most important advantages is the low cost of adding pumping and power equipment to an existing development to obtain more firm or reserve power. Three examples of such plants are given in Part II and it is the writer's opinion that considerable activity will be shown in the United States in the adaptation of existing plants to the pumped-storage principle; the adaptation of one plant has already been seriously proposed.

How far these favorable features outweigh the added investment and operating cost depends on local conditions.

Summary of Data

There are six storage plants, twenty-one poudage plants, seven part-head plants, and eight plants not classified as to type due to the scarcity of data on them. The following summary of the outstanding features of the plants built to date (1927) or under construction (see Table 1) is based on incomplete data and is, therefore, necessarily approximate:

Salient feature.	Location.
Earliest, 1882 (hydro-mechanical until 1891)	Zurich, Switzerland.
Largest reservoir, net capacity, 6 000 000 000 cu. ft. at present; 7 000 000 000 cu. ft. ultimately	Rocky River, Connecticut.
Smallest reservoir, capacity, 200 000 cu. ft.	Tübingen, Brunnenmühle, and Zweribach, Germany.
Highest head in full-head development, 2 900 ft.	Tremorgio, Switzerland.
Lowest head, 80 ft.	Cornabbia, Italy.
Highest reservoir elevation, 7 710 ft.	Rhône Valley, Switzerland.
Lowest reservoir elevation, 430 ft.	Rocky River, Connecticut.

TABLE 1.—LIST OF PLANTS.

Plant No.	Date of construction.	Location.	Type.	STATIC HEAD, IN FEET.		Reservoir elevation, in feet.	PRESENT NET RESERVOIR CAPACITY, IN BILLION CUBIC FEET.		PRESENT PUMPS.				HORSE-POWER CAPACITY OF PRESENT TURBINES.		Reservoir with or without natural inflow.	Reference numbers in Bibliography.
									Largest.		Total.					
				Power.	Pumping.		Head-water.	Tail-water.	Horse-power.	Cubic feet per second.	Horse-power.	Cubic feet per second.	Largest.	Total.		
1	1882	Zurich, Switzerland (hydro-mechanical until 1891)......	Pondage	510	510	......	0.0006	......	400	....	700	......	......	......	......	22
2	1885±	American hydro-mechanical	Pondage	......	......	......	......	......	......	....	......	......	......	......	......	No published data
3	1894	Luino, Switzerland (?)......	Pondage	......	......	......	......	......	100	....	100	......	......	......	Without	26
4	1899	Choindez, Switzerland......	......	1 970	1 970	......	......	......	......	....	......	......	......	......	......	22
5	"	Clus, Switzerland......	......	1 030	1 030	......	......	......	......	....	......	......	......	......	......	22
6	1903	Ruppoldingen, Switzerland	......	......	......	......	......	......	......	....	......	......	......	......	......	26
7	1904	Olten-Aarburg, Switzerland......	Pondage	1 030	1 030	......	0.0004	......	800	....	800	......	......	......	Without	22 and 26
8	1908	Brunnenmühle (near Heidenheim), Germany	Pondage	330	330	......	0.0002	......	160	....	240	......	......	......	Without	22 and 26
9	1909	Schaffhausen, Switzerland	Pondage	530	530	......	0.003	......	1 000	....	2 000	......	......	......	Without	2, 22, and 26
10	"	Clenezzo (near Bergamo), Italy......	Pondage	1 410	1 410	......	......	......	785	....	1 570	......	......	......	Without	22 and 26
11	"	S. El. d'Evian-Thonon-Annamasse France......	Pondage	1 310	1 310	......	0.0004	......	600	....	600	......	......	......	Without	26
12	1910	Chevenoz, Haute-Savoie, France......	Pondage	1 310	1 310	......	0.0003	......	800	....	800	......	......	......	......	22
13	"	Stura di Viú, or Funghera, Turin, Italy..	Pondage	500	500	......	0.002	0.002	4 000	....	4 000	......	4 000	......	Without	11, 22, and 26
14	"	Cornabbia, Italy	Pondage	80	80	......	......	......	5 800	....	5 800	......	......	......	Without	26
15	1913	Viverone, Novare, Italy......	Pondage	490	490	......	0.01	......	4 500	....	9 750	......	......	......	With	11 and 22
16	"	Neckartzlingen, Germany......	Pondage	410	410	......	0.006	......	320	....	540	......	......	......	Without	26
17	"	Mill Creek No. 2, Murray, Utah......	Part-head	1 030	140	......	0.0003	......	100	4	100	4	1 300	2 600	With	10
18	1915	Lake Fully, Valais, Switzerland......	Part-head	5 410	460	6 990	0.1	......	570	7	570	7	3 000	12 000	With	4, 9, 13, and 20
19	1920	Fridingen, Germany......	Pondage	570	......	......	0.001	......	620	7	1 860	21	750	2 250	......	26 and 28
20	"	Fully, Switzerland......	Pondage	490	490	......	......	......	300	....	600	......	......	......	......	26

TABLE 1.—(*Continued.*)

Plant No.	Date of construction.	Location.	Type.	STATIC HEAD, IN FEET.		Reservoir elevation, in feet.	PRESENT NET RESERVOIR CAPACITY, IN BILLION CUBIC FEET.		PRESENT PUMPS.				HORSE-POWER CAPACITY OF PRESENT TURBINES.		Reservoir with or without natural inflow.	Reference numbers in Bibliography.
									Largest.		Total.					
				Power.	Pumping.		Head-water.	Tail-water.	Horse-power.	Cubic feet per second.	Horse-power.	Cubic feet per second.	Largest.	Total.		
21	1922	Tubingen. Germany	Pondage	370	370	......	0.0002	......	190	3	....	4	290	290	Without	16, 17, 28, and 30
22	1923	Ueberlingen, Germany	Storage	360	360	......	0.05	......	580	11	580	11	600	800	With	28
23	"	Schwarzenbach (near Forbach), Baden, Germany	Part-head	1 220	730	......	0.5	......	10 000	110	20 000	220	27 500	55 000	With	7, 25, 26, and 29
24	"	Münster, Alsace, France	Pondage	1 310	1 310	......	0.0026	0.0005	1 800	9	3 190	......	2 100	2 100	Without	21 and 26
25	" ±	Ill Lake, Switzerland	Part-head	3 230	360	7 680	......	......	500	8	1 500	24	....	....	With	9
26	1924	Lake Girotte, or Belleville, Savoie, France	Storage	1 620	1 620	5 650	1.	......	4 500	16	9 000	32	....	....	With	11 and 23
27	1925	Reutlingen, Germany	Pondage	390	390	......	0.0014	0.0028	......	23	....	35	2 000	....	....	17 and 30
28	"	Wäggital, or Rempen, Switzerland	Storage	800	800	2 950	5.	......	5 000	45	20 000	180	19 000	76 000	With	8, 12, 15, and 26
29	"	Zweribach, Germany	Pondage	1 640	1 640	......	0.0002	......	1 000	5	3 000	15	780	2 340	With	3 and 26
30	"	Tremorgio, Tessin, Switzerland	Storage	2 900	2 900	6 000	0.3	......	......	....	13 000	......	12 000	12 000	With	7, 15, 18, and 26
31	1926	Rhone Valley, Switzerland	Part-head	2 930	300	7 710	......	......	500	....	1 500	......	4 200*	8 400†	....	5
32	"	Kargegg, Germany	....	410	......	......	......	......	......	....	....	......	....	....	....	26
33	"	Königstuhl (near Heidelberg), Germany	....	1 480	......	......	......	......	......	....	....	......	....	....	Without	26
34	" ±	Walchen Lake, Bavaria, Germany	....	......	......	......	......	......	24 000	....	....	......	....	....	....	16 and 27*a*
35	1927	Niederwartba (near Dresden), Germany.	Pondage	460	460	......	0.07	0.07	27 000	350	108 000	1 400	30 000	120 000	....	7, 12*a*, 24, 25, and 26
36	"	Hemfurth, Prussia, Germany	Storage	130	120	......	......	......	8 400	480	16 800	960	....	33 000†	With	26
37	"	Hengstey, Ruhr, Germany	Pondage	520	520	......	0.04	0.04	34 000	420	102 000	1 260	45 000	180 000	With	12*a*, 23*a*, and 26
38	"	Mittweida, Saxony, Germany	....	......	......	......	......	......	......	....	....	......	....	....	....	25
39	"	Munich, Bavaria, Germany	....	......	......	......	......	......	......	....	....	......	....	....	....	25
40	"	Maen, Italy	Part-head	2 730	820	6 900	......	......	......	....	....	......	....	....	With	9
41	"	Lüner Lake, Vorarlberg, Austria	Part-head	4 500	1 600	6 360	......	......	......	....	....	......	....	....	With	9 and 27*a*
42	"	Rocky River (near New Milford), Conn.	Storage	230	230	430	6.	......	8 100	250	16 200	500	33 300	33 300	With	1, 3*a*, 7*a*, 9*a*, 9*b*, and 12*b*

* Generator kilowatts. † Generator kilovôlt-amperes.

Salient feature.	Location.
Most powerful pump, 34 000 h.p.	Hengstey, Germany.
Largest pump in capacity, 480 sec-ft.	Hemfurth, Germany.
Highest pump efficiency, 87%	Rocky River, Connecticut.
Most powerful turbine, 45 000 h.p.	Hengstey, Germany.
Greatest installed turbine capacity, 180 000 h.p.	Hengstey, Germany.

Proposed Plants

The biggest project on which data have been published is that for which a grant has been applied by the Rhine-Westphalia Electric System and the General Electric Company (see Table 2). They propose ultimately to build a 600 000-kw. plant on the Our River, in Germany, with a main dam 350 ft. high, which alone will have 920 000 cu. yd. of concrete. With this large volume of concrete is to be compared the 660 000 cu. yd. at Conowingo, Md., a run-of-stream plant with an ultimate installation of 396 000 kw. At the Our River development there are three rivers, on one of which there is a reservoir with a capacity of 3 000 000 000 cu. ft. for the Stolzenberg plant which operates under a 330-ft. head. Into this reservoir the Sauer River will be diverted and here will be installed, in the power-house, pumps with an ultimate capacity of 250 000 kw. to fill the main reservoir of 28 000 000 000 cu. ft. on the Our River. The plant will replace the 25% reserve capacity of the companies' existing steam plants which will be used 6 000 instead of 3 000 hours per year, and it will produce 500 000 000 kw-hr. per year, of which one-third will be from the natural run-off and the remainder from pumped-storage. Most of the energy for pumping will be produced at steam stations.

A development of the magnitude of the Our River project indicates what, in time, may be expected in this country. An old report on Rocky River considered a 2 100-kw. installation to use the natural flow of the river, while the ultimate proposed now is twenty-three times as great, due to the utilization of the large reservoir for pumped-storage. It has been said of Rocky River that each time a greater installation of pumps was investigated the development looked more favorable, and the pumping capacity is now double that originally authorized. If this example is typical of other locations the present estimates of the country's potential water power will have to be increased many times, and the strides made in the last few years in Europe lead to the conclusion that the development of the pumped-storage hydro-electric plant in the United States will go forward at a swift pace.

II.—Description of Plants

A brief description of the most important plants will illustrate the application of the principles discussed.

Mill Creek No. 2, Murray, Utah.—This small part-head plant, built in 1913, is the first American installation on which data have been found. The striking statement has been published* that the entire cost of the pumping features added to a high-head development ($5 500), was less than the value of the additional saleable energy produced annually.

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* *Engineering Record,* March 15, 1913.

TABLE 2.—LIST OF PROPOSED EUROPEAN PLANTS.

Plant No.	Date of construction.	Location.	Type.	STATIC HEAD, IN FEET.		Reservoir elevation, in feet.	ULTIMATE NET RESERVOIR CAPACITY, IN BILLION CUBIC FEET.		ULTIMATE PUMPS.				HORSE-POWER CAPACITY OF ULTIMATE TURBINES.		Reservoir with or without natural inflow.	Reference numbers in Bibliography.
									Largest.		Total.					
				Power.	Pumping.		Head-water.	Tail-water.	Horse-power.	Cubic feet per second.	Horse-power.	Cubic feet per second.	Largest.	Total.		
1	1914	Colmar, Haut-Rhin, France	......	330	330	......	......	......	......	....	25 000*	1 270	......	......	With	22
2	1927	Chemnitz-Zwickau Region, Saxony, Germany	......	......	......	......	......	......	......	....	......	......	......	......	......	26
3	"	Leipzig, Saxony, Germany	......	......	......	......	......	......	......	....	......	......	......	......	......	26
4	"	Dresden, Germany	......	......	......	......	......	......	......	....	......	......	......	......	......	26
5	"	Struden, Germany	......	510	510	......	1.	......	......	....	......	......	......	107 000	With	19 and 26
6	"	Our River (near Stolzenberg), the Eifel, Germany	Storage	......	......	......	28.	0.3	......	....	250 000*	......	......	600 000†	With	18, 19, and 26
7	"	Siemens-Schuckert System, Germany	Pondage	820	820	......	0.004	......	6 550	56	13 100	112	10 900	443 600	Without	27
8	"	Hagen, Westphalia, Germany	Pondage	570	570	......	......	......	......	....	......	......	......	12 000†	......	26 and 31
9	"	Laacher Lake (near Namedy), Essen, Germany	......	......	......	900	......	......	......	....	......	......	......	......	With	31
10	"	Mort Lake, (near Grenoble), France	Storage	......	......	2 050	0.2	......	4 800	....	......	......	......	12 500	With	6 and 14

* Motor-kilowatts.

† Generator-kilowatts.

The main development consists of a 20-ft. dam forming a reservoir with a capacity of 300 000 cu. ft., from which the water is led by a 2.5-ft. pipe line, 27 200 ft. long, to a power-house. Two 1 300-h.p. turbines are installed at this power-house for a head of 1 030 ft. One-half mile below the dam are a number of springs with a total flow of 3 to 4.5 sec-ft., and a 4-ft. dam has been built to collect their discharge. A 100-h.p. pump, delivering 3.5 sec-ft., and operated automatically by a float-switch, pumps water against a head of 140 ft. through a 12-in. pipe into the main pipe line on the hillside above.

The operation of this plant is not entirely on the pumped-storage principle because the pumping is controlled by a float-switch and is not exclusively off-peak. At the Lake Fully plant, in Switzerland, also, the water from the pump flows up or down the penstock, depending on the load on the turbines.

Tübingen, Germany.—The reservoir for this plant, which was built in 1922, is of reinforced concrete. It is one of three which are smaller than any of the others, containing only 200 000 cu. ft., or 2 000 000 gal. The pool is 20 ft. deep and only 144 by 82 ft. in plan. Rocky River has a capacity 30 000 times as great. The reservoir is set upon a hill 370 ft. above the Neckar River, where the power-house is located, and is filled in 16 to 20 hours through a 15-in. pipe, 2 000 ft. long. In the power-house is a unit typical of present European practice even for the largest plants. It consists of a dynamo with a horizontal shaft, on one end of which is a turbine, and on the other end, a pump, the dynamo acting as a motor or as a generator as occasion demands. At Tübingen, the turbine is rated at 290 h.p. and the pump at 190 h.p., delivering 3 sec-ft. Additional pumping capacity is supplied by a pump delivering 0.9 sec-ft.

The smallness of this reservoir, taken in conjunction with those at Brunnenmühle and Zweribach, which are equally small, seems to indicate that while economic conditions are not the same in America as in Europe, it might pay some of the smaller companies in this country, when they need a few hundred kilowatts of firm capacity, to look around their hills and brooks to see whether there is not some particularly favorable location worth investigating for pumped-storage.

Schwarzenbach, near Forbach, Baden, Germany.—The adaptation of an existing hydro-electric plant to secure additional firm power makes this plant unusual. Previous to 1923 the Murg plant had been built for a head of 490 ft., with a forebay of a capacity of 10 000 000 cu. ft., an 11-ft. tunnel, 18 000 ft. long, and two penstocks, 1 000 ft. long, supplying five 7 000-h.p. turbines. For the Schwarzenbach plant these penstocks have been tapped at the bottom and water is drawn from them and pumped to a reservoir 730 ft. above the forebay of the Murg plant, or 1 220 ft. above the old tail-water level. This reservoir has a present capacity of 500 000 000 cu. ft. (600 000 000 cu. ft. in the future) for a draw-down of 130 ft. and is filled through a 6-ft. penstock, 2 600 ft. long, by two combination units, each of which has two pumps connected by gears to the main shaft. The two pumps of each unit together are rated at 10 000 h.p. and deliver 110 cu. ft. per sec. with an efficiency of 85%, the highest in Europe, while the turbines are of 27 000 h.p.

Tremorgio, Tessin, Switzerland.—This plant is an excellent example of the addition of pumping equipment at an existing development to add to its value.

Its reservoir was originally formed for pure storage by damming a natural lake and raising its surface to Elevation 6 000. Later, a 5-ft. tunnel, 600 ft. long, was driven to tap the lake and secure a capacity of 300 000 000 cu. ft. for a draw-down of 90 ft. From the tunnel a penstock 5 100 ft. long and 2.3 ft. in diameter, with provision for a second, leads to a 12 000-h.p. turbine under 2 900-ft. head in the power-house which is located on the Tessin River. This head is the highest in a full-head development. In 1926 two pumps with a total capacity of 13 000 h.p. were added, making it a pumped-storage plant. An unusual feature of this development is the tapping of the penstock to supply water to several towns.

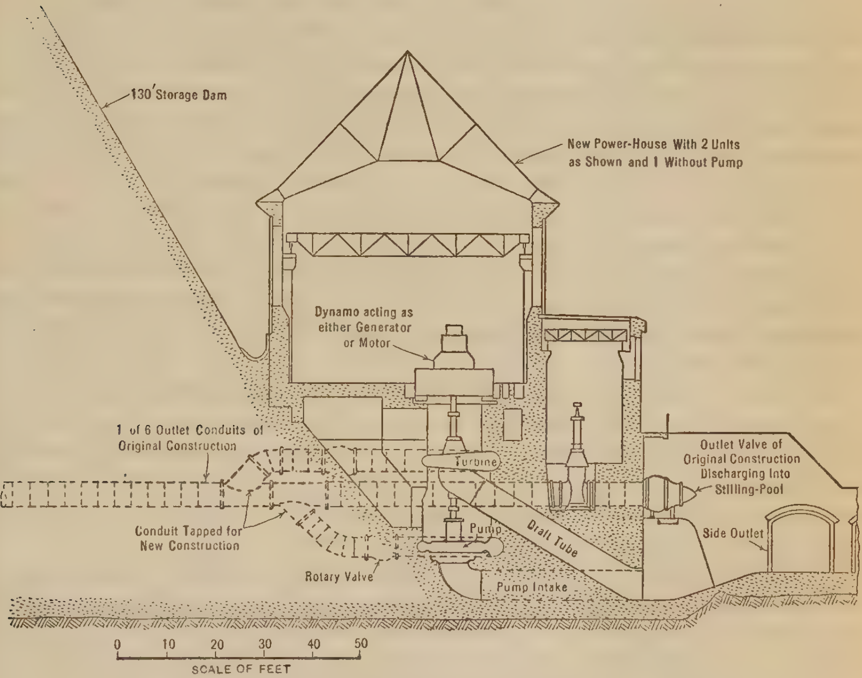
130'Storage Dam
New Power-House With 2 Units as Shown and 1 Without Pump
Dynamo acting as either Generator or Motor
1 of 6 Outlet Conduits of Original Construction
Turbine
Outlet Valve of Original Construction Discharging Into Stilling-Pool
Conduit Tapped for New Construction
Pump
Draft Tube
Side Outlet
Rotary Valve
Pump Intake
0
10
20
30
40
50
SCALE OF FEET

FIG. 4.—ADAPTATION OF STORAGE AND POWER PLANT TO THE PUMPED-STORAGE PRINCIPLE.

Hemfurth, Prussia, Germany.—This is a unique addition of pumping and power equipment to an existing development. The original construction consisted of a 130-ft. storage dam with 15 000 kw. installed in the power-house and six outlet conduits discharging through needle-valves into a stilling pool. This has been transformed into a pumped-storage plant (Fig. 4*) by tapping two of the conduits and connecting each of them by branches to a turbine-dynamo-pump unit in a second power-house. A dam has been built down stream to form a pool from which the pumps can draw water, with a fluctuation of 7 ft. in the water level.

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* From *Elektrotechnische Zeitschrift,* June 30, 1927, p. 946.

The pumps in this development have the greatest discharge of any so far built (480 sec-ft.), although their rating of 8 400 h.p. does not approach a record. The turbines are of 7 900 h.p., and the generators are rated at 6 000 kw. In addition to the two combination units there is a third unit with the pump omitted. The units are unusual in the fact that the shaft is vertical. If the plant had been designed originally as a pumped-storage plant it is possible that it could have been designed for greater hydraulic efficiency, but as an example of the application of the pumped-storage principle to a plant already constructed, it is highly interesting.

Niederwartha, near Dresden, Germany.—This is the type of plant which, in the writer's opinion, is likely to become common near metropolitan load centers in this country wherever there are hills. More than a dozen possible developments are already under consideration. The Niederwartha plant improves the operating conditions of the Dresden steam plants with which it is connected, by smoothing off the peaks and filling in the valleys of the load curve and by standing as a reserve plant available almost instantly. Two reservoirs have been constructed (an upper and a lower), each of 70 000 000 cu. ft. capacity. Water is drawn from the Elbe River near-by to fill the lower reservoir, from which it is pumped into the upper reservoir, 460 ft. above, by four 27 000-h.p. pumps. These pumps are part of combination turbine-dynamo-pump units in which the turbines are rated at 30 000 h.p. and the dynamos at 15 000 kw. The ultimate installation will be six such units, and three pipe lines have been built, 9.2 ft. in diameter and 3 000 ft. long. Each pump discharges 350 sec-ft.

Hengstey, Ruhr, Germany.—The 34 000-h.p. pumps at this plant are the most powerful yet built, and their rating is four times that of any in the United States. The 45 000-h.p. turbines at Hengstey are the most powerful in Germany, and probably in Europe; the turbine capacity of the plant, 180 000 h.p., is the largest total; and the annual output is 120 000 000 kw-hr.—yet here power is only a by-product of domestic water supply.

A settling basin was required on the Ruhr River and after consideration of various plans it was found that the capacity of the basin could economically be tripled if a pumped-storage plant of the pondage type were built. On a bluff on one bank of the river a reservoir with a net capacity of 40 000 000 cu. ft. for 45-ft. draw-down is being constructed by building an oval dike, and this reservoir is being connected with the power-house on the river by four 8.2-ft. penstocks, 1 000 ft. long. In the power-house are three combination turbine-dynamo-pump units and a fourth unit with the pump omitted. The head is 520 ft. and each pump discharges at the rate of 420 sec-ft. The sedimentation basin, which has a gross capacity of 100 000 000 cu. ft., is formed by a dam at which there is a run-of-stream plant developing a 15-ft. head. The lowering of the water level in the basin due to the draft to fill the upper reservoir, is 2.3 ft.

Rocky River, near New Milford, Conn.—This development has four outstanding features: (1) It is the first modern plant in America; (2) its reservoir is the largest of all, having a net capacity of 6 000 000 000 cu. ft. for a 30-ft. draw-down with an ultimate capacity of 7 000 000 000 cu. ft. when raised

5 ft.; (3) its elevation of 430 ft. above sea level is the lowest; and (4) its pumps have the highest efficiency, 87% being guaranteed. The lowness of the elevation of the reservoir indicates that pumped-storage is not limited to mountainous country where there is a large output from each cubic foot of water stored. As indicated in Fig. 2, Rocky River is a small tributary of the Housatonic River entering a mile above New Milford. Fig. 5* is a photograph made during the filling of the reservoir. The dam is of earth. The pumps are at the right end of the power-house. The water is led from the reservoir by a canal 3 200 ft. long and 45 ft. deep, and a 15-ft. pipe line, 1 900 ft. long, to a power-house which contains one 24 000-kw. generator and two 8 100-h.p. pumps, each delivering 250 sec-ft. Provision has been made for a second generator. The head is 230 ft. and the drainage area, 40 sq. miles, while the drainage area of the Housatonic at the power-house is 1 037 sq. miles. With two run-of-stream plants in the system the addition of firm capacity to the system by the building of the Rocky River plant will be 40 000 kw. and the additional output will be 80 000 000 kw-hr. Its regulating effect on the river will also make possible otherwise uneconomical projects.

ACKNOWLEDGMENTS

Thanks are due L. D. Gilfillan, Assoc. M. Am. Soc. C. E., for many discussions of the principles of operation with the writer, and to him and to Mr. M. S. Weil for valuable criticism. The writer wishes especially to express his deep appreciation of the criticism given by Joel D. Justin, M. Am. Soc. C. E., which has been invaluable.

APPENDIX

METHODS OF OPERATION

Power is produced from a plant of the storage type like Rocky River by the following six methods. It must be remembered, however, that only average monthly flows are considered in this discussion, variations within the months being ignored.

1.—Pondage.—Starting with full reservoir at the end of the spring freshet season, the plant is held in reserve and is idle except as explained in Sections 3 to 6. The plants of the combined steam and hydro-electric system are to be operated so as to obtain the maximum possible firm capacity from the water available. This capacity has been determined by a trial-and-error computation, in which it has been discovered that when the average monthly flow of the main river at the power-house of the pumped-storage plant has fallen to a certain quantity, it is necessary to operate the generator in the pumped-storage plant in order to maintain the firm capacity of the hydro-electric system. The unit, therefore, is operated, but all water released during the on-peak hours of the day is replaced by water pumped up into the reservoir by steam-generated power during the off-peak hours of the night and on Sunday. The firm hydro-electric capacity is thus maintained without any net depletion of the reservoir and the flow of the river is unaffected by the storage reservoir.

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2.—Flow Regulation.—When the flow of the river falls still lower, the firm capacity of the hydro-electric plants of the system is maintained at the amount determined by the computation mentioned previously, by releasing water from the reservoir to be added to the flow of the river for the benefit of the plants down stream as well as to produce power at the pumped-storage plant. This water is in addition to that which is released during the day to be pumped up again during the night (see Section 1 of this Appendix).

3.—Secondary Energy from Natural Inflow.—As the flow of the main river increases after the low-flow period the flow-regulation operation and the pondage operation of the plant are successively stopped. When the flow increases so that the pumps can draw from the river without decreasing the firm capacity by withholding water from the down-stream plants, the reservoir is refilled by pumping. Finally, when the reservoir is full the natural inflow is used to produce secondary energy and prevent the reservoir from overflowing; but otherwise the plant is idle, except as indicated by Sections 4, 5, and 6, until the next decrease in the flow of the main river, when the cycle is repeated.

In tabular form this fundamental cycle is as follows:

Flow of Main River Decreasing:

(*a*) Full reservoir.
(*b*) Secondary energy from natural inflow into full reservoir.
(*c*) Pondage operation with full reservoir.
(*d*) Pondage and flow-regulation operation drawing down reservoir with increasing rapidity.

Flow of Main River Increasing:

(*d*) (*Continued*) Pondage and flow-regulation operation drawing down reservoir with decreasing rapidity.
(*e*) Pondage operation with reservoir drawn down.
(*f*) Plant idle with reservoir drawn down.
(*g*) Refilling reservoir.
(*a*) Full reservoir again.

This cycle is reversed if at any point the tendency of the stream flow changes; that is, if in the idle period, (*f*), the flow decreases instead of increases, the pondage operation of Period (*e*) is resumed, followed by Period (*d*) until the flow again increases, when the operation goes forward on the cycle through Periods (*e*), (*f*), and (*g*).

The maximum firm capacity of the hydro-electric system is maintained by operation on this cycle, but under certain conditions this operation of the plant may be supplemented as described under Sections 4, 5, and 6.

4.—Secondary Energy with Surplus Hydro-Electric Energy.—If the plant is operated on the fundamental cycle, Sections 1 to 3, there are some parts of Period (*b*) when the reservoir is full and the plant is producing secondary energy from the natural inflow only, while water is being wasted over the dams of run-of-stream plants at night and on Sunday, the units being only partly loaded. Under these conditions coal can be saved at steam plants by drawing on the reservoir during the peak hours and refilling it by pumping with surplus

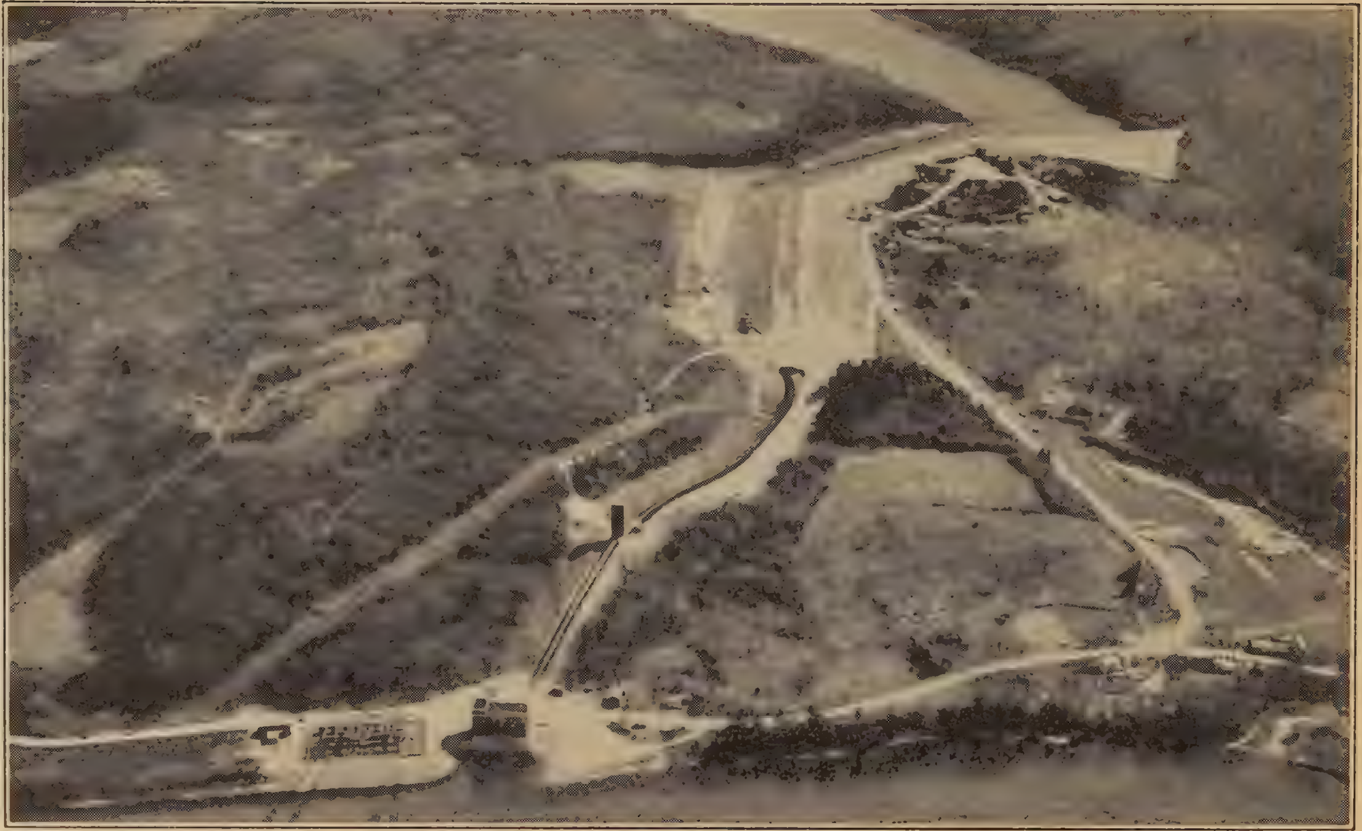

FIG. 5.- ROCKY RIVER DEVELOPMENT FROM A RETOUCHED PHOTOGRAPH MADE DURING THE FILLING OF THE RESERVOIR.

hydro-electric energy during the off-peak hours. This secondary energy from water which is replaced by pumping is in addition to the secondary energy from the natural inflow when the reservoir is full, as described under Section 3.

5.—Secondary Energy Instead of High-Cost Steam.—Again, if the plant is operated on the fundamental cycle, there may be times in Period (*b*) (while the reservoir is full) when for a few hours it may be necessary to start and operate another boiler or turbine in a steam plant. The unit cost of the energy produced is very high and, as shown in the following example, it may pay to operate the pumped-storage plant for short-time storage of secondary energy.

Assume the on-peak cost of steam-generated power under the condition described to be 8 mills and the off-peak cost, 4 mills, both at the station bus. Alternative means of supplying an additional load of short duration, with the run-of-stream plants already using all the available water and one steam turbine fully loaded, are to start either a second steam turbine or the pumped-storage plant, replacing the water used shortly afterward. Energy from either plant is delivered at the same sub-station, high-tension bus, at which point the costs may be compared.

Example.—The cost of steam-generated energy may be analyzed as follows:

Efficiencies:	Percentage.
Steam station step-up transformers	98
Relatively short transmission line to load center	95
Resultant	93

The cost at the steam station switchboard is 8 mills per kw-hr., and at the sub-station, high-tension bus, 8.6 mills per kw-hr. delivered. This cost per kilowatt-hour delivered compares with the cost of pumped-storage hydro-electric energy as follows:

Efficiencies:	Percentage.
Steam station step-up transformers	98
Relatively long transmission line to pumped-storage plant	90
Hydro-electric plant step-down transformers	98
Pump motor fully loaded	96
Pump fully loaded	87
Penstock to reservoir lightly loaded	99
After an interval of time between off-peak and on-peak:	
Penstock from reservoir partly loaded	98
Turbine partly loaded	89
Generator partly loaded	96
Hydro-electric plant step-up transformers	98
Relatively short transmission line to load center	95
Resultant	56

The cost at the steam-station switchboard is 4 mills per kw-hr., and at the sub-station, high-tension bus, 7.2 mills per kw-hr. delivered. The saving from pumped-storage instead of steam-generated energy is 1.4 mills per kw-hr., or 16 per cent. The ratio of the resultant efficiencies is 0.60.

The extent of this use of the pumped-storage plant changes with the system load, available units, cost of energy, and stream flow, and it is ignored in the financial statement of the value of the plant.

6.—Break-Down Power.—In an emergency the hydro-electric unit, if not already generating power, can be placed in operation in a few minutes. The difference, however, between the pumped-storage plant where the tail-race is a pool, and any other hydro-electric plant, is that this emergency use is not dependent on the stream-flow conditions, for the water can be retained in the pool and pumped back into the reservoir, while in other plants after the water is once released it is gone. This pumping, and pumping for pondage, are limited to the flow of the main river if there is no pool forming the tail-race.

For a plant to have definite reserve value which can be included in the financial statement there must be a big river or pool forming the tail-race and balanced generating and pumping equipment provided for operation outside the fundamental cycle.

In case there is no after-bay and the pumping is limited to the flow in the main river, the production of secondary energy at any time except when the reservoir is full, as discussed under Sections 4, 5, and 6, may jeopardize the filling of the reservoir before the next low-water period and may lower the firm capacity of the hydro-electric system.

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DISCUSSION

John R. Freeman,* Past-President, Am. Soc. C. E. (by letter).—The use of pumped storage on a very large scale was suggested about 1900 by the late Julius M. Howells, M. Am. Soc. C. E., as an important part of a development of 50 000 to 100 000 h.p. from the Feather River over a transmission line about 200 miles in length, of the Western Power Company of California.

Mr. Howells was an engineer of unusual creative imagination, as was proved by his pioneer work in the construction of large earth dams by the hydraulic-fill method. It has long been a matter of deep regret to the writer that Mr. Howells did not receive the credit due him for his inventive genius in being the first to apply the methods developed in the hydraulic mines of California to the building of large earth dams. This method has now become so well established under such widespread application that it has ceased to be thought of as an invention which required great courage, as well as technical skill in its methods, by an engineer who made use of it in important structures. The writer firmly believes that Mr. Howells was entitled to the credit for that invention.

If his associates or financial backers had possessed equal confidence and courage, Mr. Howells, long ago, would have demonstrated this method of pumped storage for electric developments on a grand scale. At the beginning of this century the United States was in the early stages of large scale hydro-electric development, and it was not then so easy to find either capital for building great plants, or customers to absorb their product.

In brief, Mr. Howells' project was to transmit the electric current at a constant 24-hour, 7-day rate to a point about 15 miles from San Francisco, Calif., thereby obtaining the greatest possible economy in the use of a transmission line nearly 200 miles in length, and to utilize all surplus power in off-peak hours for pumping the water into a reservoir about 750 ft. above sea level and ½ sq. mile in area, in the valley on the northeasterly slope of Mt. Tamalpais.

Mr. Howells' preliminary estimates assumed that the combined efficiencies of motor and pump would be about 70% during the storage period and that in the period of return the combined efficiency of the Pelton wheel and generator would also be about 70%, or that, as a whole, about 50% of the power expended in pumping would be returned.

This pumped-storage station also was to be largely relied on to take the place of the steam reserve station near the terminus, which, in those days, was considered an essential part of any long-distance transmission through rough country.

It was a fascinating proposition, but 20 or 25 years ago the time was not ripe for it. Moreover, at that time, the efficiency of multi-stage centrifugal pumps, and of impulse wheels of the Pelton type, had not become so highly developed as in recent years.

H. Alden Foster,† Assoc. M. Am. Soc. C. E. (by letter).—An interesting example of the use of pumped storage was involved in a hydro-electric project on which the writer recently made some extended studies. Since the problems treated were rather unique, it is believed that the methods used in solving them may be of interest. The writer does not claim any credit for originating the general scheme of development described.

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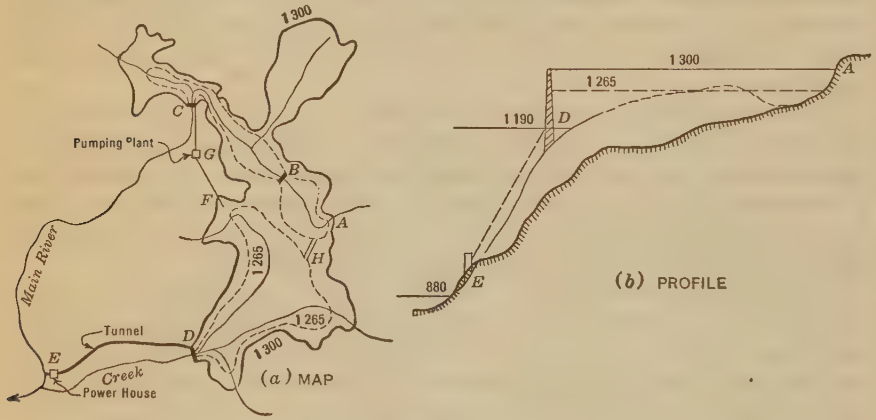
† With Parsons, Klapp, Brinckerhoff & Douglas, New York, N. Y.

Fig. 6 is a map and profile of the project. The river flows around a large bend, *A C E*, in a distance of about 14 miles. The total fall from *A* to *E* is 420 ft., as indicated on the profile. Several plans were proposed for the development of this head, one of which consisted of building a dam across the river at *C*, to raise the water level to Elevation 1 300. At this elevation, the water would overflow the main river valley into the water-shed of the creek flowing from *D* to *E*. A second dam, therefore, was located at *D*. The resulting reservoir had a flow line at Elevation 1 300 (Fig. 6 (*a*)). From *D* to *E* it was proposed to build a tunnel, which would deliver the water to a power-house at *E*, where the full head of 420 ft. could be utilized.



(*a*) MAP

(*b*) PROFILE

FIG. 6.

The reservoir itself was to be used to regulate the flow of the river down stream from *E*. Its use, however, was complicated by the fact that, when the water was drawn down below Elevation 1 265, the lake would divide into two parts, as indicated by the flow line shown in heavy broken lines on the map. The water flowing into the reservoir from above *A* was diverted into that part which was directly tributary to the tunnel, through the canal, *H*, by a weir constructed across the river at *B*, the crest of this weir being at Elevation 1 265. The only way in which the water in that part of the reservoir between *B* and *C* could be diverted to the tunnel (when the reservoir was drawn down below Elevation 1 265) was by pumping. This was arranged by locating a pumping plant at *G*, which received the water through the pipe line, *G C*, and discharged into the other part of the reservoir at *F*. The available storage was thus divided into three parts, as shown diagrammatically in Fig. 7 (*a*):

Section I.—The storage capacity above Elevation 1 265: This part of the total capacity can be drawn upon whenever the reservoir surface is above Elevation 1 265.

Section II.—The part of the reservoir capacity below Elevation 1 265 and directly tributary to the tunnel: This volume of stored water can be drawn upon whenever the reservoir surface is below Elevation 1 265. The minimum draw-down is to Elevation 1 190 (see Fig. 6 (*b*)).

Section III.—The part of the reservoir below Elevation 1 265, and lying between the weir, *B*, and the dam at *C*: This volume of storage is available when the reservoir surface is below Elevation 1 265; but it cannot be used by the power-house unless it is pumped into Section II (Fig. 7 (*a*)).

The pumping plant would operate against a static head varying from zero to 65 ft., plus a friction head based on the size of pipe line adopted and varying with the actual rate of discharge, with a maximum total head of about 80 ft. The water so pumped was available for power under a static head of between 310 and 385 ft., depending on the actual level of the water surface in Section II.

In order to determine the capacity for which the pumping plant should be designed, a mass curve (Fig. 7 (*b*)) of the unregulated stream flow was prepared, covering the dryest period in the available records. At the left of this diagram, a vertical line, *c a*, shows the three parts of the total storage, as already explained. A straight line, *a b*, is drawn tangent to the lowest hollow of the mass curve, the slope of this line being 1 400 cu. ft. per sec. This represents the maximum rate of flow that can be maintained in the river below the power-house if the total capacity of the reservoir can be utilized.

There are four methods of operation illustrated by Fig. 7 (*b*): (1) Pump water from Section III to Section II after Section I has been discharged; (2) without pumping, draw water from Section I at the rate of 1 400 cu. ft. per sec. and then from Section II at 1 200 cu. ft. per sec.; (3) without pumping, draw water from Sections I and II at the rate of 1 400 cu. ft. per sec.; and (4) without pumping, draw water from Sections I and II at the rate of 1 372 cu. ft. per sec.

By Method (1) the pumps would be operated at their full capacity at all times when Storage II is below Elevation 1 265, in order to secure the greatest possible benefit from the pumps by increasing the operating head at the power-house. The sequence of operation as shown by the mass curve is to use Storage I from *c* to *d*, and Storages II and III together, from *d* to *b*. This section of the mass curve is shown at an enlarged scale in Fig. 7 (*c*). At *b*, lay off Storages II and I vertically upward to the point, *f*. From *e*, at a distance, *I*, above *d*, draw a straight line to the point, *f*. Then, the slope of *e f* will give the maximum rate at which water can be drawn through the tunnel during low-water stages without using Storage III. The difference between the slopes of *e f* and *c e* will give the minimum discharge at which the pumping plant must operate in order to transfer all of Storage III into Storage II during the period of time, *d b*, when Storage II is being drawn upon by the power-house. The slope of *e f* is 1 200 cu. ft. per sec.; hence, the pumping plant must be designed for an average continuous capacity of 200 cu. ft. per sec. Evidently, with this method of operation, the power-house will have a continuous available draft of 1 400 cu. ft. per sec., and the three parts of the reservoir will be completely emptied at the end of the dry period.

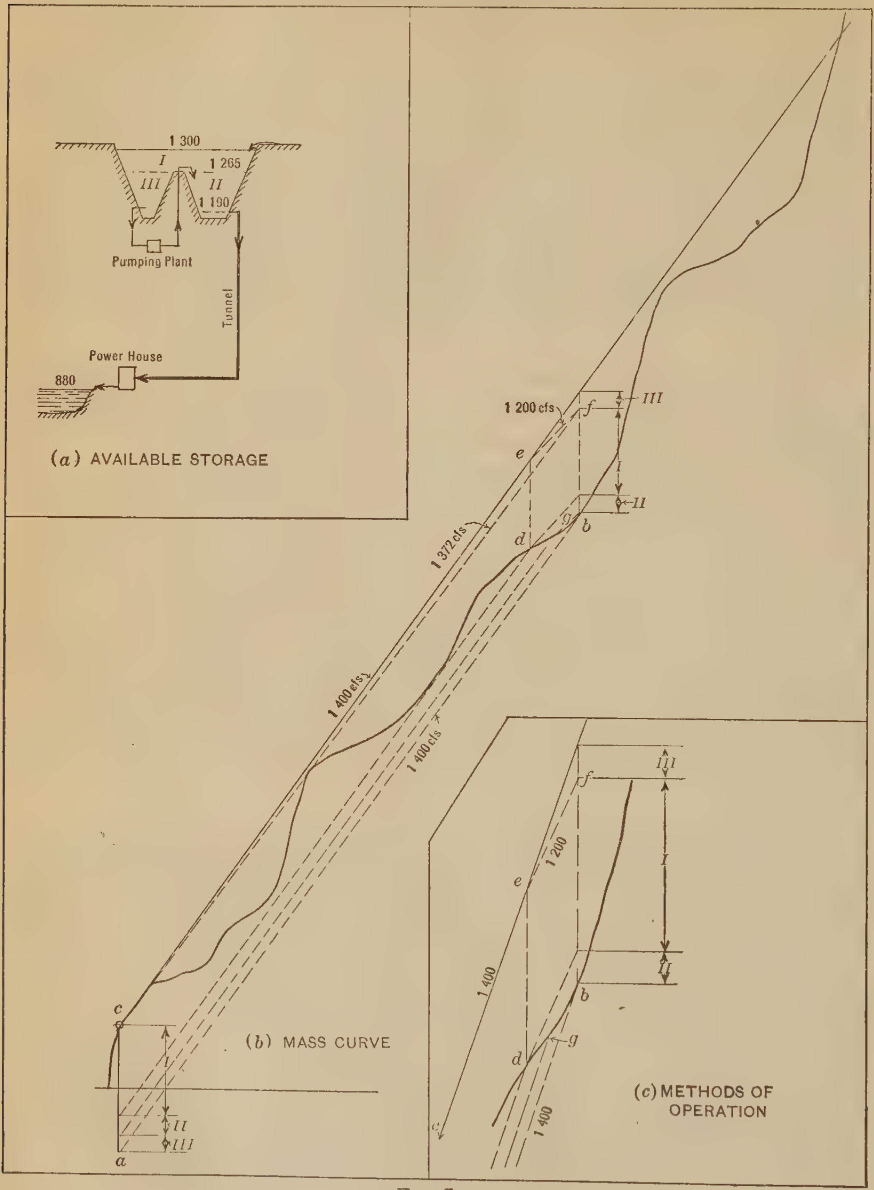

1 300
I
1 265
III
II
1 190
Pumping Plant
Tunnel
Power House
880
(a) AVAILABLE STORAGE
III
1 200 cfs
f
e
I
II
g
b
d
1 372 cfs
1 400 cfs
1 400 cfs
c
I
II
III
a
(b) MASS CURVE
III
f
I
1 200
e
II
b
g
d
1 400
1 400
c
(c) METHODS OF OPERATION

FIG. 7.

By Method (2), with no pumping plant provided, the flow through the tunnel is reduced to 1 200 cu. ft. per sec. whenever the reservoir surface is drawn down below Elevation 1 265. Storage II will then be entirely consumed by the end of the dry period, as in the case of Method (1). The primary flow at the power-house is thus reduced from 1 400 to 1 200 cu. ft. per sec., by omitting the pumping plant. The flow in the river below the power-house, however, can be maintained at 1 400 cu. ft. per sec. at all times by discharging 200 cu. ft. per sec. through the dam at *C* (Fig. 6) during the time period, *d b*. The average flow through the power-house is about 1 382 cu. ft. per sec.

By Method (3), the flow through the tunnel is maintained at 1 400 cu. ft. per sec. during the dry period until Storages I and II are all consumed (Point *g* on the mass curve). From *g* to *b*, the tunnel flow will be reduced to the natural stream flow entering the reservoir. The flow in the river below the power-house can be maintained at 1 400 cu. ft. per sec. during this period by discharging sufficient water through the dam at *C*.

By Method (4) the discharge through the tunnel is maintained at 1 372 cu. ft. per sec. at all times (slope of line, *c f*, Fig. 7 (*b*)), and no water is discharged through the dam at *C*; hence, the regulated flow in the river below the power-house is reduced from 1 400 to 1 372 cu. ft. per sec.

Under the first three methods, Storage III would be used during the dry period; but under Method (4), Storage III would never be used.

The relative advantages of Methods (2), (3), and (4), are not dependent on the pumping plant; they are mentioned chiefly to illustrate an interesting use of the mass diagram. Having determined which of these methods of operation is the most desirable, the economic justification of the pumping plant may be examined by the following procedure:

(*a*) Compute the annual fixed charges and average annual operating expense for the pumping plant.

(*b*) Compute the increase in firm capacity and average annual output, at the power-house, *E*, and at any other power-houses farther down the river, which may be obtained by the use of the pumping plant.

(*c*) Compute the kilowatt capacity required for the pumping-plant motors, and the average annual kilowatt-hours consumed in pumping.

(*d*) The difference between the capacity and energy increases of Procedure (*b*) and those required in Procedure (*c*) will represent the net increase of firm capacity and the kilowatt-hour output obtained by using the pumping plant. If the annual value of these increases is greater than the total annual cost of the pumping plant, the construction of the plant will be justified.

It will be seen that this problem is a special case of the "part-head" type of plant discussed by the author.

Oren Reed,* Assoc. M. Am. Soc. C. E. (by letter).—With favorable topography and a low-head water supply the European expedient of pumping water into a storage reservoir for use at times of peak loads, as described by the author, may be very economical and may often have a decided advantage over a steam plant designed for similar service. The most costly demand on the electrical industry is to supply the power for the peak load, which lasts, on an average, only a few hours. Great strides have been made to meet this demand by providing storage hydro-electric plants as near the load centers as possible. This storage, in many cases, is maintained partly or entirely by pumping.

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The rate of flow of some streams is similar to the power demand. This is not the case, however, in the central part of Europe, particularly in Northern Italy and Switzerland, and a proportionally greater value attaches to stored water. In these regions the head developed is usually very high, with a correspondingly small drainage area. The demand for power has greatly increased there in recent years and, on account of the high price of coal, this demand is being supplied largely by water-power plants. The streams are fed by melting snow and glaciers, and, therefore, they have their maximum flow in summer and minimum flow in winter. The present power demand, on the other hand, is greatest in winter, due to the lighting and heating load.

In the development of storage reservoirs, therefore, particular attention has been given to the possibility of supplementing the stream flow. Often plants of this type are operated only during the winter months while stream-flow plants of the same, or interconnected, system carry the summer load. As a result, it has been found to be economical to pump water from intermediate catchment basins into the primary reservoir in cases where storage is most valuable or where the release from storage is to be used through a high-head plant or a series of plants.

Because of the cheapness of brown coal in Germany, it is not economical to build hydro-electric plants, except for very favorable projects. A number of pumped-storage plants have been constructed to provide auxiliary power for the steam plants. The steam plant is run at a good load factor, and provides power for pumping at night and on Sundays. The stored water is then used during the periods of peak demand.

The efficiencies of motors, pumps, and hydraulic turbines have been brought to a point at which the losses in plants of this kind are not great. The over-all efficiency of the Herdecke pumped-storage plant in the Ruhr District, Germany, is between 60 and 65 per cent. The first plants of this type built in Germany had an over-all efficiency of less than 50 per cent.

Perhaps one of the greatest advantages of the hydro-electric stand-by would be the short time required to put the plant into service in an emergency. It could be started and synchronized with the system in a few minutes, whereas the best a steam plant could do would require several hours. Because of delay in heating boilers, the more important steam stand-bys are kept under steam at all times, and this necessitates a corresponding attendance and operating cost. Such cost might be eliminated entirely, and, in any case, would be less, if water power were substituted for steam in this service.

At some localities in Germany where the topography prevents the construction of pumped-storage hydro-electric plants, Diesel motor units are provided, as at Hamburg and Berlin. Such units have advantages similar to the pumped-storage plants, such as facility and quickness in starting in an emergency, and low charges when not in service.

In addition to the plants described by the author, the one at Tule River, California, and the Swiss plants at Arniberg, Palu, Cavaglia, and Mutten Lake, deserve mention.

Tule River, California.—The Tule River project was completed in 1914. The flow of the Middle Fork of the river is diverted by a low concrete dam into a flow-line conduit of tunnels and ditches, leading to an open forebay above the power-house. The single impulse-wheel unit operates under a static head of 1 523 ft. About ½ mile below the diversion dam there is a permanent spring near the course of the stream. In 1922, a 5-in. centrifugal pump driven by a 100-h.p. direct-connected, electric motor was installed to pump the flow from the spring to the flow-line conduit on the hillside above. The pump operates automatically on a float-switch to deliver 2.2 sec-ft. against a head of 175 ft. through 1 100 ft. of 12-in. redwood pipe. A second unit, which was designed for a 200-ft. head, was installed in 1924. Each pump operates almost continuously from May to January of each year. The net increase at the power house is estimated to be three times the power input to the pumps.

Arniberg, Switzerland.[*]—The Arniberg plant was completed in 1910. The flow of several small streams is diverted to a small forebay at the head of the penstock at Elevation 4 503. The penstock crosses Arni Brook at a point 307 ft. lower than the forebay pond. The discharge of this stream is pumped directly into the penstock. The power-house operates under a static head of 2 795 ft.

Neither of the plants described operates entirely on the pumped-storage principle because pumping is not off-peak. However, a valuable block of firm power is obtained, which amply justifies the use of the pumping plant.

Palu-Cavaglia, Switzerland.—The two plants, Palu and Cavaglia, were completed by the Brusio Power Company in 1927. The upper plant, Palu, operates from the storage in Bernina Lake, which has a useful capacity of 12 150 acre-ft. The Cavaglia plant uses the flow from the Palu Glacier, in addition to the regulated flow: A short distance below Cavaglia is the intake to the Robbia plant of the same Company, completed in 1910.

The tunnel that supplies the Palu plant leads from the south dam at Bernina Lake; it has an internal area of 31.7 sq. ft. and a length of 3 570 ft. The penstock has an internal diameter of 43.2 in. and a length of 4 130 ft. The Palu plant operates under a static head of 1 013 ft. It contains a single vertical generator of 8 000 kw., driven by two turbines on the same shaft. The main turbine is near the generator level while the auxiliary wheel utilizes the additional head of about 85 ft. between the main turbine and the equalizing pond. A plant is installed at Palu, designed for a maximum capacity of 21.2 sec-ft., to pump the summer flow from the Palu Glacier back into Bernina Lake. The flow from Bernina Lake is used in winter when the natural flow to Cavaglia and Robbia is small. The pumped storage is utilized in the three plants through a total static head of 3 703 ft. Below Robbia, there are two additional plants, Campocologno, with a head of 1 380 ft., and Poschiavino, with a 295-ft. head.

Mutten Lake, Switzerland (Proposed).—Mutten Lake, which is at an elevation of 8 020 ft., has a drainage area of only 1.15 sq. mile. It is proposed to tap the lake by a 4 920-ft. tunnel to give a static head of 5 360 ft. at the proposed power house. The normal run-off from the lake is 5 070 acre-ft. per year, while the economical regulated capacity of the lake is 15 400 acre-ft. In order to supply the deficiency, it is planned to pump water from the Limmern River into Mutten Lake against a gross head of 2 425 ft. The pumping plant would have an installed capacity of 15 000 h.p. and would use, on an average, 25 650 000 kw-hr. of summer power, which would be generated by two stream-flow plants of the same system at off-peak periods.

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[*] "Guide to Swiss Hydraulic Developments," 1926, p. 85.

[†] *Schweizerische Wasserwirtschaft,* June, 1926, p. 103, and August, 1928, p. 115.

[‡] *Loc. cit.,* June, 1926, p. 105.

Paul L. Heslop,* M. Am. Soc. C. E. (by letter).—The author is to be complimented on the painstaking thoroughness with which he has searched for, and brought to light the salient facts on the existing and proposed pumped-storage hydro-electric plants. The data gathered here illustrate the wide variety of conditions under which engineers have considered pumped-storage plants to be worthy of construction. They are not limited to high heads. (See Table 1, No. 14, Cornabbia, 80 ft.; No. 36, Hemfurth, 130 ft.; and No. 42, Rocky River, 230 ft.) The storage capacity of the reservoir and the size of the installed machinery cover a wide range of values.

Pumped-storage plants will be justified when, and only when, ordinary storage plants are not feasible or economical in the size of installation desired. An ordinary storage plant will perform all the regulating functions that can be obtained from a pumped-storage plant, with but one exception; that exception, however, is quite worthy of note. Assume that the plant discharges into a tail-water pool in which there is a reasonable storage capacity. In event of a breakdown in a steam plant of the system the pumped-storage plant may be called upon to carry part of the system load, although the reservoir may be at a stage at which withdrawal of water is not desirable. Within the limits of the effective storage in the tail-water pool the water may be pumped back into the reservoir when the steam plant is again in service, which is virtually equivalent to delaying the steam-plant breakdown to a more favorable time. Similarly, errors in the hydraulic operation of the system may be corrected within reasonable limits by pumping back the released water. With the ordinary storage plant, water once released can never be replaced as potential energy.

Pumped-storage plants in the United States will find their greatest field for consideration in the lower reaches of the various rivers, where the industrial activity is greatest, where real estate is costly, and where the valley is likely to be built up with railroads and industrial plants, making the cost of storage on the main river prohibitive. On such a location, near an important load center, a pumped-storage plant may be used to furnish peak and reserve capacity, that is, if a suitable site can be found.

In the United States pumped-storage hydro-electric plants will often have to compete with somewhat antiquated steam plants and furnish power as cheaply as the operating cost (without fixed charges) of such plants.

The author's statement that the pumping capacity of the Rocky River plant is now double that originally authorized, has an interesting explanation.

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The additional capacity was installed to reduce the time of initial filling of the reservoir and thus shorten the period before the plant would be ready for operation.

The Rocky River pumps are of the vertical-shaft, single-inlet, single-stage, bottom-suction, volute type. Prior to the construction of these pumps, a model was built and altogether twelve different combinations of impellers and suction cones were made. The pumps are by far the highest in power of the single centrifugal units in America and are, it is believed, the largest high-head pumps of any type in the country. One cannot tell from appearances that they are not hydro-electric generating units. Each pump has a suction elbow that resembles the familiar elbow-type turbine draft-tube. Each is capable of delivering 250 cu. ft. per sec., 112 500 gal. per min., or 162 000 000 gal. per day, against an operating head of 240 ft.

A. H. Markwart,* M. Am. Soc. C. E. (by letter).—Any electric generating device or method which promises to be economical in power production is of peculiar interest to the electrical utilities of the country. That attitude conforms with the enlightened policy which they have adopted, of service at the lowest cost consistent with continuous operation and sound expansion to meet the demands of present and future consumers. Naturally, an increase in the number of consumers and in quantity of power used will bring about a reduction of cost to the consumer. The electric utilities, not content with this, are always on the alert to discover ways to reduce cost, and, therefore, service rates, beyond that which would be possible by the ordinary process of mass production. To this end, much thought has been given lately to the idea of carrying peak loads on capacity units which will operate at low load factors. In this way the bottom and greater portion of system loads might be carried, in the interests of economy, on base-load plants, at load factors considerably higher than system load factors.

Methods of carrying peak loads have been studied in this country and abroad. Consideration has been given to devices for generating from a secondary source, such as electric storage batteries, steam accumulators in steam-electric generating systems, and hydraulic accumulators or pumped-storage hydro-electric plants, in both steam and hydro-electric generating systems. Consideration has also been given to devices for generating from a primary source, such as steam plants in hydro-electric generating systems, and Diesel engines. Because of their high cost, electric storage batteries have not been used extensively to carry peak loads. The development of large Diesel engines for central station use has not advanced, and it does not seem likely that the Diesel engine will be widely used to carry peak loads to improve the load factor of base-load plants. Pumped-storage plants may be suitable at times; the desiderata are (1) favorable natural conditions, and (2) a low first cost.

In the light of efforts on the part of electric utilities toward increasing economy in power production, a discussion of pumped-storage plants as a means of solving peak-load problems in electric power stations, is of timely interest. The author has grouped them into three types: (1) The pondage plant; (2) the storage plant; and (3), the part-head plant.

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Pumped-storage installations are not recognized as standard practice in America; however, wherever they are physically possible, further study may indicate economic feasibility. In fact, pumped-storage plants must be viewed largely with respect to their economic aspects because, aside from the pumping equipment involved, they present no technical features not present in the usual hydro-electric plant.

The economics relate wholly to the cost of power at the load factor under which such plants must supply power as compared with the cost from other plants, either steam or other hydro-electric, at the same load factor. One may say that pumped-storage plants, particularly the pondage and storage type, must be used, in effect, to supply kilowatts of peak load instead of kilowatt-hours of energy. Consequently, in any given system, they must work in the peak of its load, which contains a negligible quantity of energy, instead of in the base of its load which contains the bulk of the required energy. The part-head type may be conditioned so as to supply both peak and energy.

With the exception of the pumping equipment, the part-head plant has the physical composition of an ordinary hydro-electric plant, and under equal conditions of water supply the operation and character of power are the same. In comparing the cost of power from this type with that from an ordinary hydro-electric plant, account must be taken, not only of the fixed charges on the capital investment and the usual operation, maintenance, and depreciation costs, but due allowance must be made for the cost, if any, of off-peak energy used in pumping. This has the effect of adding capital not necessary in ordinary hydro-electric projects. When such off-peak energy is supplied from steam plants, its cost would be substantially that of the fuel not otherwise burned; if supplied by run-of-stream hydro-electric plants at off-peak times, its cost would be nothing. Under such conditions of operation gross output would be used in the analysis of power cost; but, if such a plant supplied its own power for pumping, the cost of power would be determined solely by the usual hydro-electric annual costs, provided net output was considered.

There might be conditions of limited water supply and favorable topography that would make it feasible to install part-head plants, operating on the part-time storage or the flow-regulation principles, to carry peaks only. The one would be a part-head pondage plant, using natural flow and ordinary forebay regulation instead of re-pumped water, and the other a part-head storage type, if there were other plants down stream and if the natural flow of the side stream were sufficient.

In order to view this subject in its essential aspect and to reduce the economic problem to its simplest terms, the writer proposes to confine his discussion to the pondage type or to any pumped-storage plant that operates on the short-time storage principle; that is, one that uses system off-peak energy, during low load periods, from other plants of the system, to generate on-peak energy during high load periods at the pumped-storage plant and at other plants down stream, if such exist.

The problem is similar to that which presents itself in the case of steam plants, operating at low load factors, which are used to comb the top of system loads that have load factors considerably less than 100% in order to permit hydro-electric plants to operate on the bottom of the load at load factors considerably greater than the system load factor. Often steam power is cheaper than water power at low load factors, and *vice versa* at high load factors. When this condition occurs, there will usually be a composite economy in power production if the system load is divided into two parts, and the bottom, or high load factor, is placed on water power and the top, or low load-factor, is placed on steam. This, of course, assumes that the capital cost of hydro-electric energy is not so low as to preclude the use of any steam and that fuel is not so cheap as to preclude the use of any water-power energy.

Because of the limitation on the quantity of water used, pumped-storage plants must operate on the top or low-load factor part of the load of a given system, where the quantity of energy to be supplied is negligible. Thus, they become capacity plants and their function is to improve the load factor of the base-load plants—hydro-electric or steam, or both as the case may be—to the end that the cost of system power will be a minimum. However, because of the remarkable over-all efficiency of modern steam plants, pumped-storage plants will be of no value as an aid to composite economy in power production unless the capital cost per kilowatt of installed capacity is relatively low. It is the low capital requirement and negligible fuel consumption that justify steam plants for the top or low-load factor load in a system using base-load hydro-electric power. The two kinds of generation facilities, that is, pumped-storage, and peak-load steam plants in a water power system, have a comparable function, and may be truly said to be in economic competition when the base-load plants are operated with water power.

In places where the topography and other natural conditions are favorable it is probable that the capital cost per kilowatt of deliverable peak, at the required, very low, load factor from a pumped-storage plant, will be enough less than that from an ordinary natural-flow hydro-electric plant, with forebay and afterbay regulation as may be required for a low load factor, to justify the installation of the former provided the off-peak power for pumping is available at a low cost. Consequently, a steam plant will usually be the alternative for a pumped-storage plant in any economic comparison; and the latter will be justified if a kilowatt of power at low annual load factors can be produced from it cheaper than from a steam plant. However, cost comparisons must be predicated on a common delivery point, since the steam plant has place utility.

In passing, the steam-accumulator electric plant might be mentioned as a possible alternative for a pumped-storage hydro-electric plant wherever the system base load is carried by steam plants. In such an arrangement the boilers of the base-load plant furnish steam for the accumulators during off-peak hours and the accumulator, in turn, supplies steam to its electric generator in order to furnish a low, load factor, on-peak energy, all in the manner in which base-load plants supply off-peak energy to elevate and store water that operates pumped-storage plants over the peak. The steam accumulator is peculiarly adaptable in steam-electric generating systems; consequently, it is improbable that it would ever be the best alternative for a pumped-storage plant in which the base load is carried on hydro-electric plants. For the purpose of a generalized economic study and to make this discussion applicable to either hydro-electric or steam-electric generating systems, the ordinary steam plant, which is suitable in either system, has been chosen as the alternative for a pumped-storage plant.

When a pumped-storage plant is being considered for a system that requires additional capacity, it should be expected to carry only that part of the peak which would bring about a nice balance of system on-peak and off-peak energy, with due regard for the over-all inefficiency of the transmission, pumping, and generating combination. The part of the peak of new load which such a plant should carry can be determined only by making an intensive study of the load curve of the system in which it will operate.

This will furnish the basic information necessary to determine the quantity of off-peak energy which is available in an existing system at various times, and the amount of on-peak energy to be supplied when different amounts of the peak are to be carried by pumped-storage plants. Likewise, it furnishes a means of measuring the annual cost of off-peak energy to be used for pumping in case fuel is used in its generation.

Two composite load curves similar to those used in other economic studies* are submitted in this connection. One is for a system load factor of 65% (Fig. 8), such as would obtain for regional systems in California, and the other for one of 45% (Fig. 9), such as would obtain for a metropolitan system, perhaps anywhere. For convenience, these curves were based on a system load of 400 000 kw. They are, in effect, composite daily load curves, 365 of which would contain, with a great degree of accuracy, all the kilowatt-hours that would be found under 365 actual daily load curves. They are necessarily inaccurate with respect to the 400 000-kw. peak because there would not be 365 peaks of such a magnitude. There is an appearance of error in the number of kilowatt-hours lying in the peak, because there would be few such peaks, perhaps only one, in the year. The degree of error would seem to be measured by the number of kilowatt-hours in about 364 such peaks lying in that part of the load above the average annual peak; but this seeming error does not exist in the composite curves, because the method used in constructing them took into account this very thing. Thus, in effect, the curves, while not being the exact shape of daily load curves, are such that they will reflect with a high degree of accuracy the kilowatt-hours that must be produced annually for a load of 400 000 kw. at 65% annual load factor for regional power systems in the one case, and at 45% for metropolitan systems. Similarly, the curves are sufficiently accurate to ascertain the kilowatt-hours in various divisions of the load to base load and peak load and the off-peak kilowatt-hours in the base load. In Fig. 8 the dotted division line between the base-load plant and the peak-load plant is at 68.6% of the peak-load ordinate and, in Fig. 9, it is at 48.5 per cent. The energy ratio, off-peak to on-peak, equals 1⅔ : 1, which is equivalent to a pumped storage over-all efficiency of 60% in both cases.

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* "Power in California," by A. H. Markwart, M. Am. Soc. C. E., *Journal*, Franklin Inst., Vol. 204, 1927, p. 179.

To illustrate the use of the curves: If, in Fig. 8, 100% of the peak from the base, or 400 000 kw., is carried on steam, or on hydro-electric plants capable of operating at a load factor of 100%, these plants will supply 100% of the energy lying under the load line, or 2 277 600 000 kw-hr. at the system annual load factor; and by utilizing water otherwise going to waste, in the case of hydro-electric plants, or by burning additional fuel, in the case of steam plants, they would be capable also of supplying additional energy equal to 53.85% of the required system energy, or 1 226 400 000 kw-hr.

	ABOVE LOAD		LOAD			
			From Base		From Peak	
% Peak	L.F.	% KWHR	L.F.	%KWHR	L.F.	%KWHR
100	50.9	53.85	65.0	100.00	0.0	.0
95	47.0	46.35	68.5	99.95	0.7	0.05
90	42.8	38.83	72.2	99.77	1.5	0.23
85	38.0	31.51	76.2	99.39	2.7	0.61
80	32.9	24.79	80.2	98.41	5.2	1.59
75	28.1	19.03	83.8	96.47	9.2	3.53
70	24.7	14.84	86.5	92.96	15.3	7.04
65	22.7	11.90	88.3	88.20	21.9	11.80
60	19.6	8.77	90.8	83.63	26.7	16.37
55	17.1	6.33	92.8	78.37	31.3	21.63
50	14.3	4.18	94.8	72.82	35.4	27.18
45	11.0	2.37	96.8	66.93	39.1	33.07
40	8.2	1.14	98.3	60.46	42.8	39.54
35	6.2	0.38	99.5	53.52	46.6	46.48
31	0.0	0	100.	47.61	49.4	52.39
30	0.0	0	100.	46.07	50.2	53.93
25	0.0	0	100.	38.40	53.5	61.60
20	0.0	0	100.	30.71	56.4	69.29
15	0.0	0	100.	23.04	59.0	76.96
10	0.0	0	100.	15.36	61.2	84.64
5	0.0	0	100.	7.68	63.2	92.32
0	0.0	0	100.	0.0	65.0	100.00

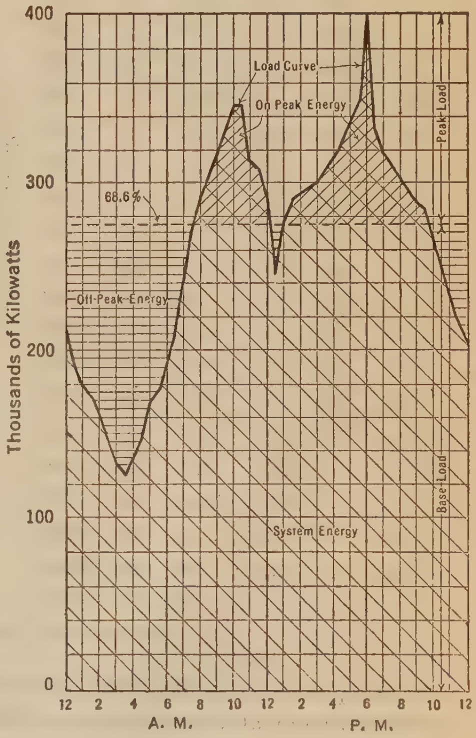

FIG. 8.—ANALYSIS OF COMPOSITE DAILY LOAD CURVE AT 65% ANNUAL LOAD FACTOR.

Similarly, if 60% of the system peak from the base, or 240 000 kw., is to be carried on like base-load hydro-electric or steam plants, and the other 40%, or 160 000 kw., is to be carried on pumped-storage hydro-electric plants, the latter would have to supply 16.37% of the system energy, or 373 000 000 kw-hr. The off-peak energy available for pumping purposes would be the equivalent of 8.77% of the system energy, or 200 000 000 kw-hr. Obviously, such division of load between the pumped-storage plants and base-load plants would not be feasible, because the on-peak energy is less than the off-peak energy. Fig. 8 shows an approximate equality, 11.90% against 11.80%, when 65% of the load is carried on base-load plants and 35% on pumped-storage plants. This is also an unbalanced condition because there is no margin of off-peak energy to make up for the inefficiency of the pumped-storage plant combination.

%	ABOVE LOAD		LOAD			
			From Base		From Peak	
Peak	L.F.	% KWHR	L.F.	% KWHR	L.F.	% KWHR
100	64.6	122.00	45.0	100.00	0.0	.0
95	62.3	110.94	47.3	99.96	0.4	0.04
90	60.0	99.99	49.8	99.81	0.8	0.19
85	57.2	89.11	52.6	99.59	1.3	0.41
80	54.2	78.34	55.7	99.26	1.7	0.74
75	50.8	67.73	59.2	98.77	2.2	1.23
70	46.9	57.33	62.8	98.07	2.9	1.93
65	42.5	47.34	67.0	96.96	3.9	3.04
60	37.8	37.77	71.3	95.43	5.1	4.57
55	32.3	28.71	76.3	93.39	6.6	6.61
50	26.9	20.91	80.9	90.09	8.9	9.91
45	24.3	16.20	83.7	83.70	13.3	16.30
40	21.9	12.19	86.0	76.61	17.5	23.39
35	19.3	8.58	88.7	69 12	21.3	30.88
30	16.2	5.39	91.7	61.21	24.9	39.79
25	12.4	2.75	94.7	52.75	28.3	47.25
20	6.0	0.67	98.2	43.73	31.6	56.27
15	0.0	0	100.	33.41	35.2	66.59
10	0.0	0	100.	22.27	38.8	77.73
5	0.0	0	100.	11.15	42.0	88.86
0	0.0	0	100.	0.0	45.0	100.00

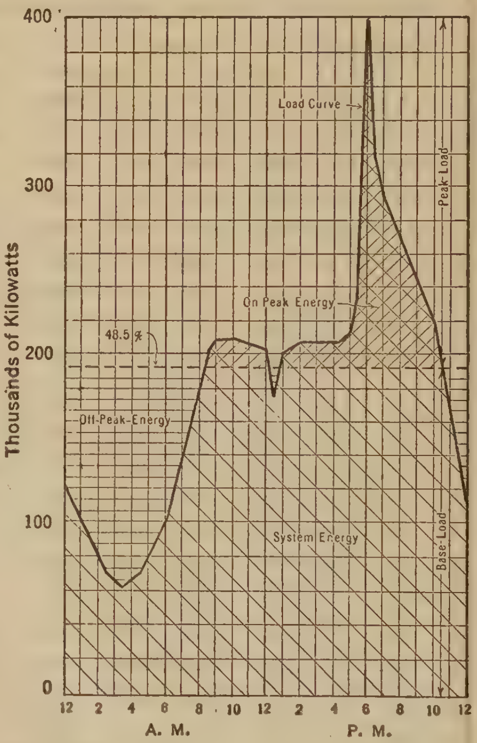

FIG. 9.—ANALYSIS OF COMPOSITE DAILY LOAD CURVE AT 45% ANNUAL LOAD FACTOR.

It is probable that the efficiency of transmission, pumping, and generation from the point at which the off-peak kilowatt-hours are available to that at which the on-peak kilowatt-hours are to be generated at the pumped-storage plants, will be about 50%, although 60% is possible. If 60% efficiency is chosen, 1 kw-hr. of generated on-peak energy would require the use for pumping, of $1\frac{2}{3}$ kw-hr. of off-peak energy. Consequently, the balance would be found when the quotient, resulting from dividing the percentage of off-peak kilowatt-hours by the percentage of on-peak kilowatt-hours, is $1\frac{2}{3}$. Then all the off-peak energy would be absorbed and no waste would occur. At 65% of the peak from the base such a quotient is approximately 1.1; at 70%, it is approximately 2.1; so, it seems that the balance is between the two. It actually occurs at approximately 68.6% of the peak from the base and 31.4% of the peak from the top as determined graphically by Fig. 10 (*a*). With this division, the equivalent of 13.8% of the system energy would be available for pumping water to generate on-peak energy to the extent of the required 8.3% of the system energy. The load factor of the top would be 17.2 per cent.

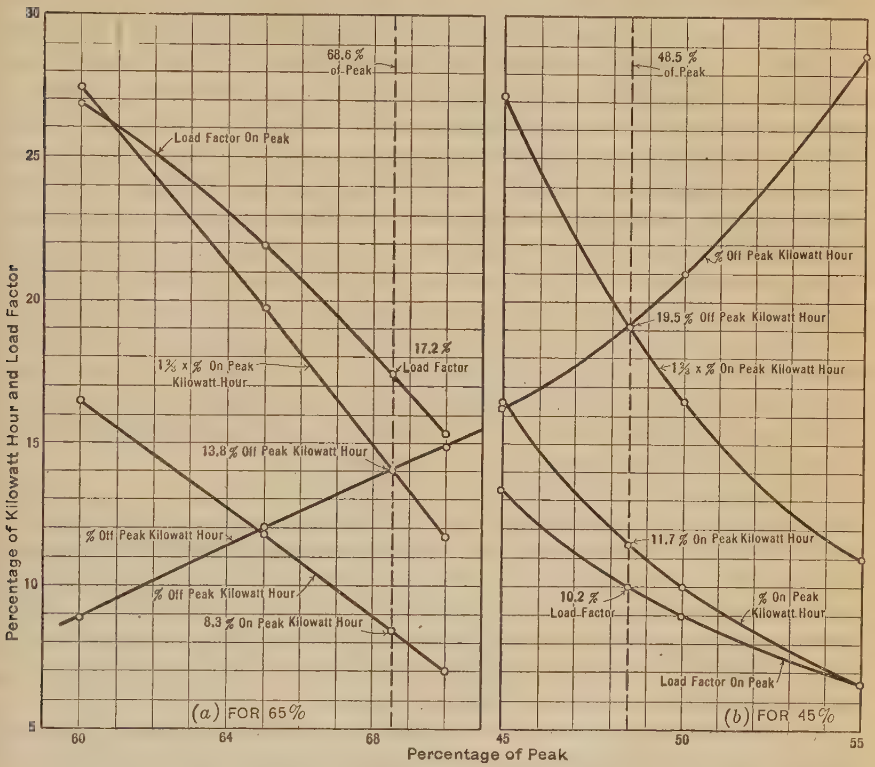

FIG. 10.—ANNUAL LOAD FACTOR SYSTEM.

A similar scrutiny of the 45% load-factor curve of Figs. 9 and 10 (*b*) reveals that a balance of off-peak with on-peak energy would be obtained by dividing the kilowatt load in two parts: 48.5% to base-load plant and 51.5% to pumped-storage plant. This division would cause the off-peak energy to be 1⅔ times the on-peak energy and would cause no waste of off-peak energy. There would then be available the equivalent of 19.5% of the system energy for the purpose of pumping water to generate on-peak energy to the extent of the required 11.7% of the system energy. The load factor of the top would be 10.2 per cent.

It would appear, therefore, that if 65% and 45% represent the upper and lower limits, respectively, of load factors likely to be encountered in actual systems with triangular peaks, substantially all the off-peak kilowatt-hours of base-load plants can be balanced against on-peak kilowatt-hours of pumped-storage hydro-electric plants. However, due allowance is made for the over-all inefficiency of the pumping-storage combination when 31.4% of the peak for a 65% annual system load factor, and 51.5% of the peak for the 45% annual system load factor, are carried on pumped-storage plants. This would leave the remainder of the load in each case to be carried on water power, or steam, base-load plants.

The curves of Figs. 8 and 9 give other interesting information: For example, in the case of the 65% load-factor load, if the pumped-storage and base-load plant generation capacities are assumed to be 31.4 and 68.6%, respectively, of the load to be carried, motorization to the extent of approximately 37.6% (68.6% base load minus 31% minimum system load) of the load to be carried, or 20% more than generation, would be necessary. Thus, if all the off-peak energy is to be absorbed, the pumping load determines the capacity of the electrical unit when it is to have the double function of generator and motor. Likewise, the pumping load determines the capacity of the transmission line if one line can serve the dual purpose of bringing off-peak power to the pump and taking on-peak power from the generator. In the 45% load factor case, if the respective figures are 51.5% and 48.5% for pumped-storage and base-load plant generation capacities, the motorization will be 48.5% — 15%, or 33.5 per cent. In this case, generation would determine the capacity of the electrical unit, and also that of the transmission, if single transmission obtained.

Of course, the information relating to percentages of system peak available for pumping is of a general order only. It can be used as a guide to explore daily load curves for individual days to determine the relation of the off-peak and on-peak electrical capacities. However, the load of the minimum as well as the maximum day, was included in making up Figs. 8 and 9. These composite curves are offered for what they are worth in suggesting a method to aid economic study only, rather than to determine the exact relation between motorization and generation for design purposes.

Another fact of some interest is revealed by reading the curves together. The quantity of off-peak energy that is available for pumping under conditions of practical balance between the two kinds of energy is reduced as system load factors rise. This is indicated by the 13.8% of the system energy that is available at the low-load period in the 65% load factor case as compared to the 19.5%, with a 45% load factor. From this it would appear that pumped-storage plants are more suited to metropolitan systems which have low system load factors, than to regional systems which enjoy high load factors. Furthermore, there is that other feature, which tends to bar their general use in steam-supported regional hydro-electric systems which usually enjoy high load factors, relating to the control of the water to suit the load factor of the load.

Often it is possible to regulate the flow at the plants of regional systems that operate on stored water so that plants on streams that have no storage can carry block load. Steam plants designed to carry peaks and a number of hydro-electric plants of varying output characteristics permit this form of operation, with the result that no appreciable quantity of water is wasted during low-load periods, if there is a proper ratio of steam to hydro-electric energy in a given situation. Some waste of water would occur if a hydro-electric development were introduced too soon or if the growth of load was less than the anticipation. Usually, however, the steam plants care for any growth in the load, and the bringing in of a new hydro-electric plant is timed in accordance with its capacity, so that it will be fully used when it takes over the load accumulated for it by the steam plants. Illustrative of this is the system of the Pacific Gas and Electric Company. It consists of thirty-two hydro-electric plants, five major steam plants, and an extensive network of power lines which serve the northern and central parts of California. The peak load and output in 1928 were 484 000 kw. and 2 545 713 564 kw-hr., respectively. This system has no off-peak hydro-electric energy and, generally, it has been possible to control the construction program so that there is substantially no off-peak energy. The spilled hydro-electric energy in the spring does not exceed 3% of the total generated output.

The system has steam energy which, under some conditions, might be available for pumping at times when the steam plants are not being operated over peaks. On the other hand, the steam plants serve the same economic purpose as pumped-storage plants and they have the additional function of supplying stand-by against a water power or transmission failure. Furthermore, they supply energy to make up the water power deficiency in years of reduced run-off (when conditions are reversed and the steam plants are operated on the base of the load and the water power plants on the peak). Therefore, they can hardly be considered as means for supplying the energy necessary to the operation of pumped-storage plants. In fact, even if the base load of this system were carried on steam plants it is highly probable that pumped-storage plants would be uneconomical because the water power sources that might be suited to such projects are at a considerable distance from the localities where steam plants are situated. However, it might be possible, in regional systems that have a metropolitan load, to utilize a pondage type of pumped-storage plant, providing a pump-storage site was available in close proximity to the steam plant in the metropolitan area, and that the economy in power production for the remainder of the regional system was not affected adversely.

The conditions that make pumped-storage hydro-electric plants economically feasible will vary in each locality; individual situations must be analyzed. The cost of peak power at the required low load factor from other available sources is the determining factor, and it is probable that usually steam power will be the alternative for pumped-storage power, with off-peak power from either steam or run-of-stream hydro-electric plants.

However, it is possible to state with approximate correctness, a general economic rule for pumped-storage hydro-electric plants under the following six premises:

(1) The total additional load to be carried will be divided between the pumped-storage peak-load plant and the base-load plant (either run-of-stream hydro-electric or steam). Under these conditions, the base-load plant will be

of a capacity such that when operating at a load factor of 100%, it will produce off-peak energy enough to balance the on-peak energy of the pumped-storage plant without substantial waste.

(2) In an economic analysis the capital cost and fuel economy of the alternate steam plant shall be (*a*) that of a plant having the capacity of the pumped-storage plant, in cases where the base load can be carried on run-of-stream water power; and (*b*) that of a plant with a capacity equal to the total additional load to be carried, in cases where the base load must be carried on steam.

(3) The marginal cost of off-peak steam power will be for energy fuel only. (The fuel economy must be that of a base-load steam plant with a 100% load factor and a capacity equal to the difference between that of the pumped-storage plant and the total load to be added to the system, with no consideration of stand-by fuel.)

(4) The marginal cost of off-peak hydro-electric energy will be zero. (Under such an assumption it is obvious that knowledge of the capacity and capital cost of the base-load hydro-electric energy is unnecessary.)

(5) Only pumped water is available to the pumped-storage plant.

(6) Nothing except on-peak power is produced as a result of the installation of pumped-storage.

Under these six premises the economic rule would be as follows: The economic limit for capital expenditure for pumped-storage plants is the capitalized annual charges of steam plants operating under similar load conditions, as follows:

> 1.—In the event that the off-peak energy is supplied by run-of-stream hydro-electric plants the limit will be the annual cost of a kilowatt-year of equivalent steam power, capitalized at the annual percentage cost for hydro-electric plants.
>
> 2.—In the event that the off-peak energy is supplied by steam plants, the limit will be the annual cost of a kilowatt-year of equivalent steam power minus the annual cost of the fuel burned to supply a kilowatt of pumped-storage peak, capitalized at the annual percentage cost for hydro-electric plants.

It may be objected that these statements are mere truisms and are, therefore, of no significance, but truisms are significant if they cause the basic elements in what appears on its face to be a confused problem, to be universally recognized. Generalized statements of the elementary principles which attach to the problem under discussion are desirable in order to simplify its solution. The endeavor has been made to formulate such statements.

It is obvious that the allowable capital expenditure per unit of capacity for pumped-storage plants, for each price of fuel, will depend on the fuel economy and capital cost of the alternate steam plant, and also on the fuel economy under which the off-peak energy is generated if it is supplied from steam plants. The values of these elements, in turn, depend on the size of the plant. Consequently, economic comparisons must be made with like capacities and concomitant conditions. The capital cost per unit of capacity of steam plants and the fixed charges, that is, the interest, depreciation, taxes, operation, and maintenance cost, and the cost of stand-by fuel, or the annual charges except for energy fuel, decrease as the capacity increases; but the fixed charges, expressed as a percentage of the capital cost, will be substantially the same for all capacities of plant, varying only slightly for different unit prices of fuel. (See Table 3.)

In the case of a small pumped-storage plant a relatively high capital expenditure per kilowatt of capacity would be justified because its alternate steam plant would have a high capital cost and a relatively low fuel economy. On the other hand, the justifiable capital expenditure per unit of pumped-storage capacity for a large plant would be relatively low because the alternate steam plant in that case would have a low capital cost and a relatively high fuel economy.

It is probable that the total operating charges on pumped-storage plants would be close to those which would obtain for ordinary hydro-electric plants, and when expressed as a percentage of the capital cost, would be substantially the same for all capacities of plants.

An economic study under the rule will be made for a range of capacities from 5 000 to 100 000 kw., assuming that the annual fixed charges for steam plants are 15.5%, 15.75%, and 16% of the capital cost for $1.00, $1.50, and $2.00 oil, respectively, and that the annual fixed charge for pumped-storage plants is 10.25 per cent.

The annual fixed charge for a steam plant which is the alternate for pumped storage, operating in conjunction with base-load hydro-electric power, should be a little greater than that of the steam plant which is the alternate for pumped-storage operating in conjunction with a base-load steam plant. A little more stand-by fuel would be used in the first case, because it would be necessary to bring the alternate steam plant into operation each day from cold boilers in order to carry the load for a period of not more than two shifts, or about 16 hours. This kind of operation would be unnecessary in the latter case, because the alternate steam plant would then be a part of the larger capacity that would be required to pull the total additional load, and it would be operated continuously instead of intermittently. However, this element is disregarded in the studies, because it is more or less academic; the effect, when considered, would not have been perceptible in the final result.

The fuel economies for the chosen capacities of alternate steam plants and base-load steam plants are intended to be those which would obtain with plants designed for moderate steam pressures and elevated temperatures ranging from 400 lb. with 700° total heat for the larger plants, to 250 lb. with 650° for the smaller ones. These conditions are generally used in steam plant practice and for that reason power resulting from such plants, rather than steam power resulting from the greater economies which would obtain with plants designed for elevated pressures and corresponding temperatures, is considered the alternate for power from a pumped-storage plant. However, in places where a pumped-storage plant of very large capacity is a possibility a steam plant with a 1 200 lb. capacity may be the alternate. In that case the study might properly be based on a fuel economy of about 11 500 B. t. u. per kw-hr., as compared with 13 000 B. t. u. (which is the highest value

TABLE 3.—RELATIVE COST OF VARIOUS STEAM PLANTS.

Steam plant capacity, in kilowatts.	Capital investment per kilowatt, in dollars.	Interest, depreciation, and taxes at 12%, in dollars per kilowatt per year.	Operation and maintenance, in dollars per kilowatt per year.	STAND-BY FUEL: OIL AT 6 250 000 B. T. U. PER BBL.: COAL AT 12 500 B. T. U. PER LB., OR 4 TO 1.									TOTAL COST PER KILOWATT PER YEAR OF STEAM POWER, NOT INCLUDING COST OF ENERGY FUEL AND ANNUAL PERCENTAGE COST ON INVESTMENT PER KILOWATT.					
				Millions of British thermal units per kilowatt per year.	Barrels of oil per kilowatt per year.	Tons of coal per kilowatt per year.	Cost per kilowatt per year, in dollars.						$1.00 oil or $4.00 coal.		$1.50 oil or $6.00 coal.		$2.00 oil or $8.00 coal.	
							$1.00 oil.	$4.00 coal.	$1.50 oil.	$6.00 coal.	$2.00 oil.	$8.00 coal.	Dollars.	Percentage.	Dollars.	Percentage.	Dollars.	Percentage.
5 000	175	21.00	5.00	6.00	0.960	0.240	0.960		1.44		1.92		26.96	15.40	27.44	15.67	27.92	15.96
10 000	155	18.60	4.25	5.80	0.928	0.232	0.928		1.39		1.86		23.78	15.35	24.24	15.64	24.71	15.95
20 000	130	15.60	3.60	5.50	0.880	0.220	0.880		1.32		1.76		20.08	15.45	20.52	15.79	20.96	16.12
40 000	110	13.20	3.00	5.25	0.840	0.210	0.840		1.26		1.68		17.04	15.48	17.46	15.87	17.88	16.25
60 000	104	12.50	2.65	5.10	0.816	0.204	0.816		1.22		1.63		15.97	15.34	16.37	15.75	16.78	16.14
100 000	100	12.00	2.50	5.00	0.800	0.200	0.800		1.20		1.60		15.30	15.30	15.70	15.70	16.10	16.10

chosen for this study), except that no material change should be made in capital cost, because the investment per unit of capacity does not change as materially with changes in pressure and temperature, as it does with a change in size. The economic position of pumped-storage capacity would be slightly modified if a British thermal unit value per kilowatt-hour, based on the greatest advancement in the art of designing steam plants, were chosen in the studies. In most individual instances, this type of alternate steam plant would probably justify a smaller capital expenditure for pumped-storage plant than the moderate pressure type.

Assume that the pumped-storage plant or its alternate steam plant will generate at 17.2% load factor in the case of the 65% load factor, and at 10.2% load factor in the 45% case. This would seem to be necessary (see Figs. 8 and 9) if the division of the load is to be such that it will bring about a balance between on-peak and off-peak kilowatt-hours after making allowance for losses. The energy requirements per kilowatt per year would be 1 507 and 893 kw-hr. for on-peak generation and 2 511 and 1 489 kw-hr. off-peak pumping, respectively, for the two cases. The quantity of energy fuel to meet this condition of operation for the various sizes of plants, used in this study, is given in Table 4.

TABLE 4.—QUANTITY OF ENERGY FUEL REQUIRED FOR PLANTS OF VARIOUS CAPACITIES.

Capacity of plant, in kilowatts.	British thermal unit required per kilowatt-hour.	Kilowatt-Hour Generated.		Fuel per Kilowatt Capacity.							
				With 17.2% Load Factor.				With 10.2% Load Factor.			
		Per barrel of oil.	Per ton of coal.	To generate 1 507 kw-hr. of on-peak energy per kilowatt of peak.		To generate 2 511 kw-hr. of off-peak energy per on-peak kilowatt.		To generate 893 kw-hr. of on-peak energy per kilowatt of peak.		To generate 1 489 kw-hr. of off-peak energy per on-peak kilowatt.	
				Oil, in barrels.	Coal, in tons.	Oil, in barrels.	Coal, in tons.	Oil, in barrels.	Coal, in tons.	Oil, in barrels.	Coal, in tons.
4 700	15 600	400	1 600	....	......	....	......	....	......	3.72	0.931
5 000	15 520	403	1 612	3.74	0.935	....	......	2.21	0.552	....	......
9 400	14 800	421	1 684	....	......	....	......	....	......	3.54	0.885
9 700	14 780	422	1 688	....	......	....	......	2.12	0.530	....	......
10 000	14 740	423	1 692	3.56	0.890	....	......	2.11	0.528	....	......
10 900	14 640	425	1 700	....	......	5.91	1.478	....	......	....	......
15 900	14 220	439	1 756	3.43	0.858	....	......	....	......	....	......
18 800	14 040	445	1 780	....	......	....	......	....	......	3.35	0.837
19 400	14 020	446	1 784	....	......	....	......	2.00	0.500	....	......
20 000	13 990	447	1 788	3.37	0.843	....	......	2.00	0.499	....	......
21 800	13 910	449	1 796	....	......	5.59	1.398	....	......	....	......
31 800	13 630	458	1 832	3.29	0.823	....	......	....	......	....	......
37 700	13 530	462	1 848	....	......	....	......	....	......	3.22	0.804
38 800	13 510	463	1 852	....	......	....	......	1.93	0.482	....	......
40 000	13 500	464	1 856	3.26	0.815	....	......	1.93	0.482	....	......
43 700	13 440	466	1 864	....	......	5.39	1.347	....	......	....	......
56 500	13 280	472	1 888	....	......	....	......	....	......	3.15	0.789
60 000	13 230	473	1 892	3.18	0.795	....	......	1.88	0.470	....	......
63 700	13 200	474	1 896	3.18	0.795	....	......	....	......	....	......
77 700	13 100	478	1 912	....	......	....	......	1.87	0.467	....	......
87 400	13 040	479	1 916	....	......	5.24	1.310	....	......	....	......
94 200	13 010	480	1 920	....	......	....	......	....	......	3.10	0.776
100 000	13 000	481	1 924	3.13	0.782	5.22	1.305	1.86	0.465	3.10	0.776

The economical limits of capital expenditure for pumped-storage as determined by the rule are given in Fig. 11 (*a*) and Fig. 11 (*b*). The data for the curves were determined by using the cost of power from alternate steam plants with different prices for fuel. It is assumed that the off-peak pumping energy is supplied either by run-of-stream hydro-electric plants or base-load steam plants and that the annual fixed charge for the pumped storage is 10.25 per cent.

While this study is not precise and obviously cannot cover an individual situation, it sets with sufficient accuracy the upper limit of capital expenditure for pumped-storage plants of the capacities considered when they are to operate on the top of 65% and 45% system loads having triangular peaks, if the cost of power therefrom is to be just equal to that of similar power from fuel plants. However, pumped-storage plants would not be economically attractive if the power from them was just equal in cost to alternate steam power; it would have to be considerably cheaper than steam power to offset the disadvantage which attaches to a plant that can be used only to "pull" peak loads. A steam plant can always furnish energy up to its capacity by burning sufficient fuel. Consequently, one would not build a pumped-storage plant in lieu of a steam plant unless the estimated capital cost of the pumped-storage plant was well below that which would bring an equality of cost with steam power as indicated by economic study; or unless, where base load is carried by both run-of-stream hydro-electric and steam plants, enough useful secondary power resulting from natural inflow to pumped-storage reservoir or from water pumped by surplus hydro-electric energy, was produced to reduce substantially the quantity of fuel otherwise burned in the base-load steam plants.

Savings resulting from pumped-storage operation in any individual situation may be measured by the difference between the upper limit of capital expenditure per kilowatt of pumped-storage capacity and the estimated cost per kilowatt of capacity in a given instance. For example: Consider a system that has an annual load factor of 65 per cent. Suppose that an increase of 60 000 kw. in capacity is contemplated; that a pumped storage site is available at which power can be developed at a capital cost of $200 per kw. of peak; that the base load will be carried on hydro-electric plants; and that the current price of coal is $6 per ton. Under such a set of conditions the 60 000 kw. of additional capacity would be divided into 18 840 kw. to pumped storage and 41 160 kw. to additional base-load water power. The former would operate at a load factor of 17.2% and the latter at 100 per cent. The upper limit of capital expenditure for off-peak energy from run-of-steam hydro-electric plants is $253 per kw., as given by the curve for the $6 fuel in Fig. 11 (*a*). Thus, the saving per kilowatt per year less than what steam peak power would cost at 17.2% load factor is 10.25% of the difference between $253 per kw. and $200 per kw., which is the estimated capital expenditure for the pumped storage, or $5.43 per kw. per year. If this saving, $102 300 annually, is sufficient to overcome the disadvantages inherent in pumped-storage plants, and not present in steam plants, the pumped-storage plant would be justified.

Similarly, with a system load factor of 45%, off-peak power from base-load steam plants and an estimated cost of $200 per kw. for the pumped-storage plant, the additional capacity would be divided into 30 900 to pumped storage and 29 100 to base-load steam; the former would operate at a load factor of 10.2% and the latter at 100 per cent. The upper limit of capital expenditure as given by the $6 coal curve, off-peak energy for steam plants, is $140 per kw. (Fig. 11 (*b*)). In this case the excess cost of pumped storage over alternate steam power is 10.25% of the difference between $200 per kw., the estimated capital cost, and $140, the limiting capital expenditure, or $6.15 per

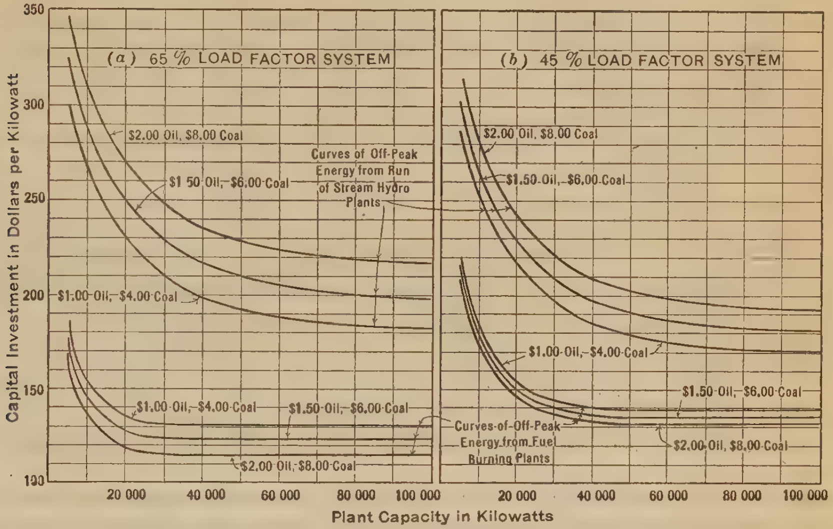

FIG. 11.—ECONOMICAL LIMIT OF CAPITAL EXPENDITURE PER KILOWATT OF INSTALLED CAPACITY.

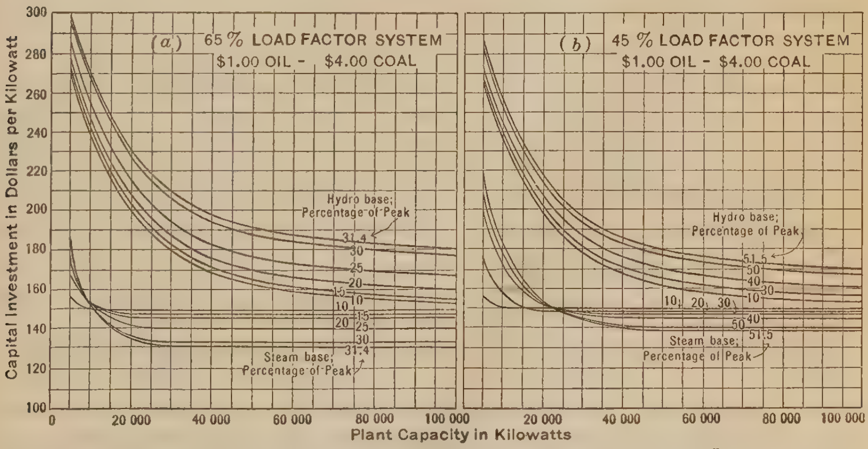

FIG. 12.—ECONOMICAL LIMIT OF CAPITAL EXPENDITURE PER KILOWATT OF PUMPED-STORAGE HYDRO-ELECTRIC PLANTS FOR VARIOUS CAPACITIES AND DIFFERENT PERCENTAGES OF PEAK.

kw. per year ($190 000 per year). Obviously, this would be an uneconomical development.

Thus far, the discussion has dealt with the maximum amount of peak load that could be carried by pumped-storage plants in the two load-factor cases on an annual basis. However, smaller amounts might properly be carried if for one reason or another it was not feasible to bring about a balance between the annual on-peak and off-peak energy of a given load to be added to a system. It is convenient to plot other curves, hydro-electric off-peak energy below and steam off-peak energy above the maximum curves, which would represent the limit of pumped-storage capital expenditure for various percentages of the peak less than the maximum percentage at which

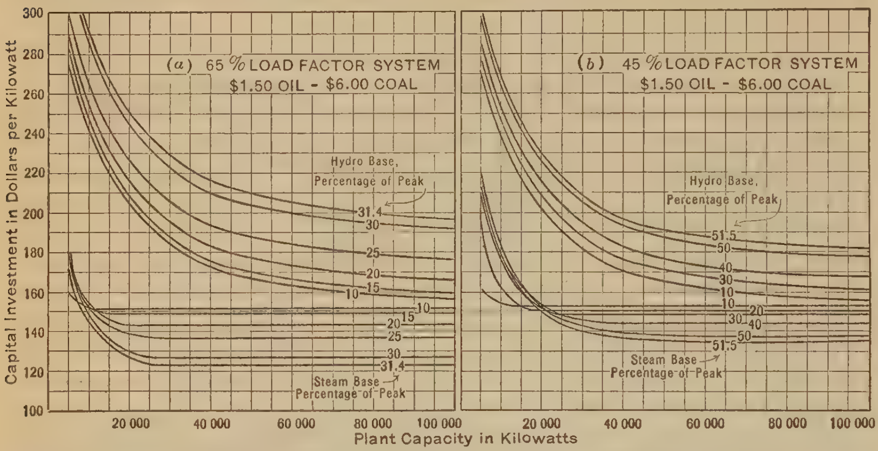

FIG. 13.—ECONOMICAL LIMIT OF CAPITAL EXPENDITURE PER KILOWATT OF PUMPED-STORAGE HYDRO-ELECTRIC PLANTS FOR VARIOUS CAPACITIES AND DIFFERENT PERCENTAGES OF PEAK.

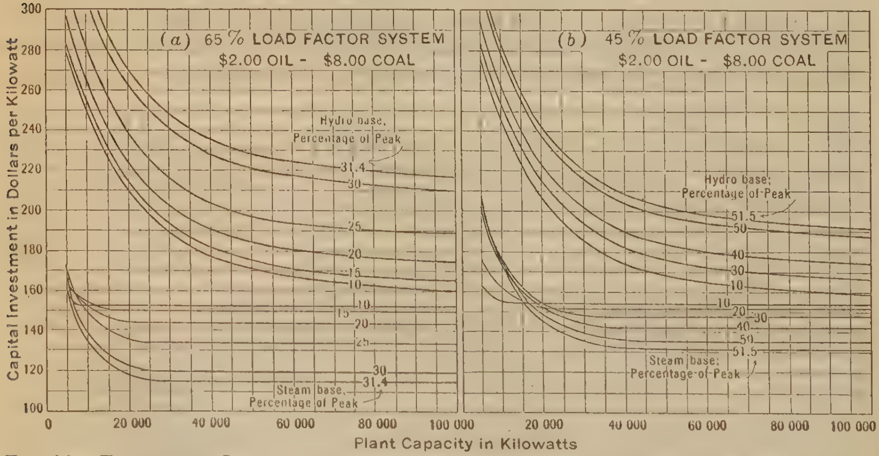

FIG. 14.—ECONOMICAL LIMIT OF CAPITAL EXPENDITURE PER KILOWATT OF PUMPED-STORAGE HYDRO-ELECTRIC PLANTS FOR VARIOUS CAPACITIES AND DIFFERENT PERCENTAGES OF PEAK.

annual off-peak and on-peak energy balance obtains. Tabulations are not included herein in this connection, but curves are given in Figs. 12, 13, and 14, for the two load-factor cases and for the three prices of fuel. These curves can be used in a manner similar to that discussed in connection with those in Fig. 11. It must be borne in mind, however, that any "combing" of the peak with pumped storage that is less than that which would bring about a balance of the annual off-peak and on-peak kilowatt-hours would cause the base-load plant to operate at a load factor of less than 100% on some days.

Regarding this last point it is improbable that a pumped-storage base-load plant arrangement could ever be such as to permit the base-load plant to operate at an annual load factor of 100 per cent. The reason for this is that on many days of the year the daily load factor would be greater than the annual load factor and there will be a day which has a greater number of kilowatt-hours than there are in the annual composite day which might be used solely for the purpose of broad economic study. In such a day of maximum energy the division of load to peak and base as determined for the annual composite day would not be the proper division of load in order to effect for the maximum energy day, the desired on-peak and off-peak kilowatt-hour relation. This situation causes a change, perhaps slight, in the economic picture as presented, when using a composite annual load curve. Consequently, a correction must be made to cover this condition.

This is illustrated in Fig. 15, which is a representative part of a 12-month composite daily load curve covering May, June, and July, 1928. The basic data are the same as those used in Figs. 8 and 9. The January curve is the composite January day, the February curve, the composite February day, and the other curves (through May, June, July, etc., to December) are corresponding composite monthly days.

The composite load curves illustrated in this study are made by plotting the half-hourly loads for the period considered (either a year or a month) on a 24-hour base downward during the hours when the load is decreasing, and upward during the hours when the load is increasing. There were plotted 1 440 ordinates for the composite curve for a 30-day month (Fig. 15) and 17 520 ordinates for the composite curve for a year (Fig. 8). The composite load curve has been found more useful than the more commonly employed "load duration curve" constructed by re-assembling in the order of their decreasing magnitude, the ordinates representing the half-hourly loads for the period under consideration, because the composite curve actually looks like a load curve and closely approximates many of the daily curves; it reveals the distribution of the load throughout the day, which the "load duration curve" does not. For convenience, the ordinates of the composite monthly curves are plotted in percentage of the yearly peak.

By extending the curve in Fig. 15 to include the entire year, the day that would be likely to have the greatest number of kilowatt-hours can be determined by inspection. It would appear to be the composite June day because its peak is less than the maximum, being 95% of it, and the minimum of this composite June day is one of the highest, being 33% of the peak. As a matter of fact, a study of the actual records of the system load for 1928 reveals that the greatest number of kilowatt-hours was generated on June 13, and that the load factor for the day was 74.56 per cent. The actual half-hourly loads of that day, also expressed as percentages of the peak for 1928, were plotted adjacent to the composite June day. Areas *A* and *B*, Fig. 15, which are representative of off-peak and on-peak kilowatt-hours, respectively, for this day of maximum energy, were measured. It was found that the 1 to 1⅜ relation was established when 75.8% of the annual peak was placed on the base for this day as contrasted with the 68.6% as determined by the study of the annual composite daily load curve, characteristic of the system in question (see Figs. 8 and 9). The on-peak energy above the 75.8% line, projected across the twelve composite monthly days, is equivalent to that above the 75.8% line on the annual composite curve in Figs. 8 and 9. The load factor and percentage of kilowatt-hours from peak will be 8.4% and 3.10%, respectively. These relations are to be determined by the use of a diagram similar to that given on Fig. 10.

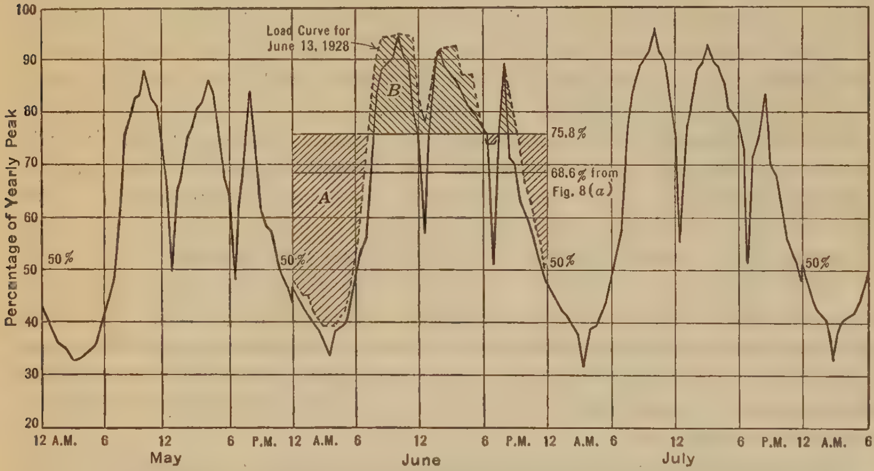

FIG. 15.—REPRESENTATIVE PART OF COMPOSITE DAILY LOAD CURVE FOR EACH MONTH.

The additional curves in Figs. 12, 13, and 14, are particularly useful in that they afford means of determining the economic situation respecting pumped storage if a study of the load curve of an individual day indicates that more load must be carried that day on the base-load plant than is indicated by the study of the annual composite daily load curve. The necessity for considering the maximum energy day becomes apparent when it is remembered that the curves of Fig. 11 are based on a composite daily load curve for the entire year, no actual day of which will be exactly like this composite curve. Moreover, if the curves of Fig. 11 were precisely interpreted they would indicate that storage could be carried over from week to week or month to month in a given year, and this would probably not be the case in actual practice. Curves such as those in Figs. 12, 13, and 14, will then indicate what economic correction, if any, is desirable when load conditions of the critical day and off-peak energy conversion are to be taken into account entirely on a daily basis.

To make a practical application of these curves, with reference to the example, previously used (60 000 kw. additional load): The division of load would be 24.2%, or 14 520 kw., to pumped storage, and 75.8%, or 45 480 kw., to base load. By reference to Fig. 13 (*a*) and the hydro-electric base-load curve for 25% of peak from the top, it will be seen that the capital expenditure for a plant of 14 520 kw. capacity should not exceed $242 per kw. as contrasted to the $253 per kw. given in Fig. 11 (*a*). Were such division of the 60 000 kw. additional load to be made, the pumped-storage plant would carry a variable peak throughout the year, operate at an annual load factor of 8.4, and supply 3.10% of the system kilowatt-hours. This detail is given to indicate a procedure which might be used to determine the capital limitations of pumped storage with greater refinement, if the maximum daily load factor is substantially more than that of the annual composite load curve which might have been used in an over-all economic study. The example chosen indicates that the limit of capital expenditure for pumped storage should be reduced about 5% less than that determined without giving consideration to the conditions which obtained for the maximum-energy and load-factor day and those which would prevail under daily instead of period carry-over storage operations.

The curves in Figs. 12, 13, and 14, reveal a number of interesting facts relating to the economics of such peak-load plants:

1.—Hydro-electric off-peak power warrants a larger capital expenditure than steam off-peak power.
2.—The limit of capital expenditure decreases:
    (*a*) With increase in fuel price when off-peak power is supplied from steam plants; and
    (*b*) With decrease in fuel price when off-peak power is supplied from run-of-stream hydro-electric plants.
3.—As the load factor decreases, the limit of capital expenditure:
    (*a*) Increases when off-peak power is supplied from steam plants; and
    (*b*) Decreases when off-peak power is supplied from run-of-stream hydro-electric plants.
4.—The limit of capital expenditure decreases with increasing plant capacity:
    (*a*) With no tendency to become a constant for capacities under 100 000 kw. when off-peak power is supplied from run-of-stream hydro-electric plants, irrespective of the system load factor, the percentage of peak to be pulled, and the price of fuel;
    (*b*) With a tendency to become a constant for capacities of more than from 10 000 to 55 000 kw. when off-peak power is supplied from steam plants, depending on the system load factor, the percentage of peak to be "pulled", and the price of fuel.

The economic position of pumped storage depends largely upon the efficiency of the several elements of its physical composition, and the economy of production of off-peak energy. The first point needs no discussion, but with respect to the second some further comment is in order. The writer has assumed that the total additional load is to be carried by pumped-storage and by modern base-load plants rather than by pumped-storage and old base-load plants. This consideration is of small economic importance in cases where hydro-electric plants are used on the base, because off-peak energy from such plants is regarded as having no value; but it is a factor when the off-peak energy is supplied from old steam plants that have a low economy. Very little capital could be spent per unit of pumped-storage capacity in the latter case, because of the large amount which would be deducted under the rule to allow for the cost of the off-peak energy. This becomes apparent when it is remembered that with many old steam plants the economy is about 30 000 B.t.u. per kw-hr., and because only a few of the old installations have been replaced with modern plants with high efficiencies, the average for all plants of the country is about 20 000 B.t.u.

It is probable that the capitalized value of off-peak energy based upon such economies would reduce the capital which could be spent per unit of pumped-storage capacity, so as to make impossible the building of such plants. However, if physical conditions were so fortunate as to make it possible to build a pumped-storage plant within a capital sum per unit of capacity that was determined on the basis of using low-economy off-peak energy, the reduction in cost per kilowatt per year (when the off-peak energy was finally supplied on a high-economy basis) would equal the difference between the low and high-economy fuel costs of off-peak energy per kilowatt per year. However, this is not likely to occur when very low-economy off-peak energy is available. For example, a fuel economy of 30 000 B.t.u. per kw-hr., with about $4 coal, would cause the cost for off-peak energy to be about $12 and $7 per kw. per year, respectively, for the 65% and 45% load-factor loads, in turn requiring pumped storage, in order to compete with alternate steam, to be procurable at from $60 to $100 per kw. It seems improbable that pumped storage could be built at a cost of $100 per kw., or less, when that must include all transmission, pumping, generation, and storage facilities.

Low-economy off-peak power will occasionally be used with pumped storage, despite the probability that most pumped storage is likely to be barred if off-peak energy is regularly to be supplied by old steam plants. For example, it might be quite inconvenient to build a complement of base-load steam at the time when it is most necessary to build pumped storage, which estimates showed could be physically accomplished, provided the capital allowance was established on the basis of supplying off-peak energy from a high-economy steam plant. In such a situation the pumped-storage plant might properly be constructed, and for a time operated with off-peak power from old steam plants, with no disadvantage, save that the cost of energy from the pumped-storage plant would be high during the interim pending the completion of the new steam plant. The excess in cost per kilowatt per year would equal the difference between the low and high-economy fuel costs per kilowatt-year.

The best position for pumped storage with steam-electric power is that in which the base load is carried on new high-economy steam plants. This element, that is, the efficiency of operation of the base-load plants, is not important when the base load is carried on run-of-stream hydro-electric energy, because then, since off-peak energy is considered valueless, pumped storage would have its greatest opportunity and much of it could be built which otherwise might not be feasible where the base load is carried on steam.

Owing to the fluctuating price of fuel, judgment must be exercised in selecting the fuel cost to be used in an economic study. To be on the safe side it would be the part of wisdom, when off-peak power is to come from steam plants, to choose a long-time price of fuel of more than that which might obtain at the time a pumped-storage plant was being considered, if then the price was low; and when off-peak energy is to come from run-of-stream hydro-electric plants, one that was less than that prevailing at the time, if then the fuel price was high. This reasoning follows from the fact that a lowering of the economic limit of capital expenditure for pumped storage, in cases where off-peak energy is supplied from steam plants, comes with an increase in the price of fuel, but if the off-peak energy is supplied from run-of-stream plants, it comes with a decrease in the price.

This generalized study of the economics of pumped storage is methodical only and is intended to be suggestive rather than dogmatic. There are doubtless other ways to portray the situation, particularly if the cost of pumped storage is known. The endeavor herein has been to set up something without a knowledge of the pumped-storage capital costs. Other cost assumptions will show different generalized results. However, it is believed that the curves of capital limit indicate orders of capital cost for pumped-storage plants that cannot be materially exceeded if the resulting cost of power is to be no greater than the current value of power as established by modern, fairly high-economy steam plants. It should not be inferred that this analysis will obviate the necessity for intimate study in individual situations, but it will be helpful in identifying the pumped-storage projects which, in connection with appropriate base-load plants, are worthy of intensive study to determine the cost of mingled energy from a complement of such plants. The method may be used for the purpose of generalizing with other load factors and under the same or other assumed data.

It is apparent to those in the power industry that one must be critically discerning when a determination is to be made in the matter of future power supply. What is desired is the lowest-cost electricity, irrespective of whether it comes from hydro-electric or fuel-burning plants, or from a combination of both. All recognize that, with annual-system load factors which are always less than 100%, there will be an annual economy if the narrow peaks of the system load can be carried by plants which can be built at a very low capital cost per unit of capacity. This applies to almost any kind of a peaking plant.

The future course of power development necessary to carry total loads is, in general, not fully established. At present, steam power appears to be more or less in the ascendant because fuel is cheap and fuel economies are high. On the other hand, the abundance of cheap capital and the economic advantages which accrue by the use of high transmission voltages, causes hydro-electric energy to remain a source of power not to be overlooked, particularly when it is remembered that the price of fuel may be higher in the case of steam power, and sufficient experience has not been accumulated to determine with certainty the cost of operation and maintenance of modern steam plants; whereas the fixed charges which make up the major costs of hydro-electric power remain fairly stationary over a relatively long period of years. Thus, the time to build hydro-electric plants is when capital is cheap and it is sound business, if capital is readily obtainable, to use hydro-electric power when its cost is equal to or less than that of steam power. However, the economic picture in the hydro-steam scene is a moving rather than a fixed one; consequently, the engineer must be cognizant of economic conditions generally, in order that he may shift from one to the other or to both of these major power sources in delving for reduced costs. Because of the rapidly changing conditions, sound economic analysis is more important in the field of electric generation than it has been heretofore. In conclusion, be it said to the credit of the civil engineer that he is getting a broad grasp of the electric industry and is being increasingly recognized by it as a factor in the field of economic energy production.

William W. K. Freeman,* Assoc. M. Am. Soc. C. E. (by letter).—In his discussion, Mr. John R. Freeman cites the proposal made by the late Julius M. Howells, M. Am. Soc. C. E., about 1905, to deliver power to San Francisco at a weekly load factor of 100% so as to cut down the investment in 200-mile transmission lines, fitting this power to the load curve through the action of a pumped-storage plant. This is one of five special applications of pumping which have been proposed to take up the slack between supply and demand. A second application is to the case of a navigable river where no pondage is permissible, in which case off-peak power may be converted into on-peak power in a similar manner. A third example is the proposed use of surplus blast-furnace or coke-oven gas to operate gas pumps of large capacity to store water in the off-peak hours.

In contrast to these cases in which power is available at a constant rate, two other special applications have been proposed to take power when it is available at variable times and deliver it to the load as required. One of these is being planned by Dexter P. Cooper, M. Am. Soc. C. E., at Haycock Harbor, Me., in connection with the Passamaquoddy Bay tidal project. The pumped-storage part of the project involves a reservoir with a net capacity of 13 000 000 000 cu. ft. for 40-ft. draw-down, with a maximum static head of 140 ft. The plant would operate on a monthly cycle, with 195 000 h.p. of pumping and 240 000 h.p. of turbine capacity. The advance beyond present record installations is shown by the proposal, among others, of one manufacturer for combination units, each unit with a turbine of 48 000 h.p. capacity and a pump of 39 000 h.p., delivering 2 700 sec-ft.

The other application is in connection with the development of wind-power plants. While no pumping feature is under consideration in the plans for the first wind-power plant, the engineers developing the plant have recognized the possibilities of a combination of wind power and pumped storage to convert off-peak into on-peak power.

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* Asst. Engr., New England Power Constr. Co., Boston, Mass.

The discussion by Mr. Foster is valuable in pointing out the use of the mass diagram (Fig. 7) in the solution of pumping problems, and because he looks upon the addition of pumping equipment as one of a number of possible methods of developing power in a certain locality, and gives a clear picture of its economics.

Mr. Reed has added descriptions of four plants to the list of those built. The writer was already indebted to him for his description of Lüner Lake and three others.* Mr. Reed's emphasis on the stand-by feature of pumping plants in Germany is in line with the point of view in this country for some of the proposed developments. The rapid progress of interconnection in the United States, however, probably removes this development of pumping plants to a later date when the older steam plants which are now held in reserve, will have been scrapped.

Mr. Heslop also emphasizes the reserve value of pumping plants. As a matter of fact, the first use of the Rocky River Plant was as a reserve plant, because as soon as the operating engineers of The Connecticut Light and Power Company were informed that the plant was ready for limited service, even before the reservoir was filled, the operation of reserve boilers at the Devon Steam Plant was discontinued.

Mr. Markwart's discussion is exceptionally valuable because it includes curves of justifiable investment cost for plants of the pondage type. These curves can be used as guides to determine which projects are worth further investigation and which are obviously uneconomical. The writer's experience, while limited, bears out the statement that, "because of the remarkable over-all efficiency of modern steam plants, pumped-storage plants will be of no value as an aid to composite economy in power production unless the capital cost per kilowatt of installed capacity is relatively low".

The curves, Figs. 11, 12, 13, and 14, show the limiting cost to be about \$120 to \$150 per kw. for systems with base-load plants burning coal that costs, respectively, \$4 to \$8 per ton. In October, 1929, Professor Dr.-Ing. D. Thoma, Director of the Hydraulic Institute, Technische Hochschule, Munich, gave the limiting cost in Germany as \$80 per h.p., equivalent to \$113 per kw., which checks roughly Mr. Markwart's figures.

Since projects costing only \$120 to \$150 per kw. are rare, Mr. Markwart's calculations bear out the writer's opinion that pumping plants of the pondage type do not appear to be as attractive as those of the storage or part-head types, unless perhaps they are built in conjunction with existing developments. It is the storage feature of Rocky River which makes it economical in spite of the comparatively low total head through which the stored water is used; this head averages 285 ft., excluding old hydro-mechanical developments. The Rocky River plant has several other advantages, moreover, which help to make it economical; the water supply for its pumps is drawn from a drainage area of 1 037 sq. miles, and the power house is as close to the load center as is the base-load steam plant at Devon; that is, about 30 miles. Certain features of topography are also in its favor;

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* See p. 902, Reference (9).

the reservoir basin was partly swamp and abandoned farm land, and the fall of the Rocky River is largely concentrated into the mile above its mouth, so that the total volume of earth in the dam and dikes is only 750 000 cu. yd.

The exceptional value of plants of the pondage type for emergency service may in time make them more attractive, but at present capacity is "a drug on the market" in many localities because of the rapid progress of interconnection which has made many small steam plants available for stand-by service.

The writer was able to check points on Mr. Markwart's curves of justifiable investment cost by following the usual method of comparison with an alternate steam plant, as outlined by the writer and amplified by Mr. Markwart under his six premises. Mr. Markwart states that his study is methodical only, and warns against using his curves for individual situations. This warning the writer would emphasize by adding that even for a certain power system at least one change in the method of analysis appears to be necessary.

This relates to the sizes of hydro-electric and steam plants assumed to be alternative. Mr. Markwart states that,

> "In the case of a small pumped-storage plant a relatively high capital expenditure per kilowatt of capacity would be justified because its alternate steam plant would have a high capital cost and a relatively low fuel economy."

Throughout his discussion of capacities, steam pressures, and fuel economy, Mr. Markwart assumes that these must be selected for a steam plant of the same size as the pumping plant under discussion. This is true only in the broadest generalization, because the selection of capacity for a pumping plant depends essentially on the topography, while selection of capacity, etc., for a steam plant depends on the characteristics of the system and the rate of growth of its load. Thus, for a system in which 50 000-kw. steam units are being installed, the capacity, steam pressure, fuel economy, operation and maintenance charges, and load factor must be that of the 50 000-kw. alternative steam unit whether the pumping plant is of 5 000 or of 200 000 kw. The difference in interest charges on unused capacity until the larger of the alternative plants is fully loaded, is applied to the construction cost in arriving at the justifiable investment.

Another change in the method of analysis which would be necessary for a particular case would be the inclusion of an increment operation and maintenance cost in addition to the coal cost given by Mr. Markwart in Table 4, in calculating the energy charges of an installation. This cost is made up of coal storage and boiler and turbine maintenance; it equals about 0.3 mill per kw-hr. in a plant of 250 000 kw. capacity, and probably more for smaller plants. A closer analysis would also require the selection of economies, as given in Table 4, not only for size of plant, but also for load factor, instead of using one figure for everything from 10 to 100% load factor.

In conclusion, there are a few matters to be mentioned to bring the subject more nearly up to date. The Rocky River pumps, which were guaranteed to have 87% efficiency, have been tested since installation and a maximum efficiency of 91.9% was obtained with both pumps running under a dynamic head of 230 ft. This is far more than the previous maximum of 85% obtained at Schwarzenbach, Germany.

The term, "pumped-storage plant," has found a certain measure of acceptance. The ambiguous term, "hydraulic accumulator," has been observed only in a digest of a German article translated too literally. Other terms which have been proposed as a substitute for the writer's "pumped storage" (which is a free translation of "Pumpen-Speicherkraftwerk") are: (1) "Water storage battery"; (2) "hydro-regenerating plant"; and (3) "pumping plant". The first of these might be considered ambiguous because of its possible association with a wet-cell electric storage battery, and the second lacks the brevity of the third. The writer inclines to "pumping" instead of "pumped storage" because of its brevity, where it is obvious what kind of pumping plant is meant.

Three important additions to the bibliography should be noted. Joel D. Justin, M. Am. Soc. C. E., has presented a paper* entitled, "The Rocky River Plant of the Connecticut Light and Power Company"; and Paul L. Heslop, M. Am. Soc. C. E., a paper† entitled, "A Hydro-Electric Plant That Pumps Its Own Water Supply—The Rocky River Hydro-Electric Plant." Mr. Heslop has also written an article‡ on the Rocky River pumps entitled, "Two 8 100 H.P. Pumps Supply Water for Power Generation".

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* *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 690 (Abstract).

† *Proceedings*, Connecticut Soc. of Civ. Engrs., 1928, p. 6.

‡ *Power*, December 24, 1929, p. 1006.

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Paper No. 1747

THE GRAPHICAL SOLUTION OF A CORRELATION TABLE*

By Weston S. Evans,† Esq.

With Discussion by Messrs. Walter H. Dunlap, Robert C. Strachan, S. L. Moyer, Theodore Hatch, B. F. Jakobsen, and Weston S. Evans.

Introduction

To the engineer the graphical solution of any problem is interesting and welcome. It also seems that the solution of a correlation table‡ for Pearson's product-moment coefficient is to him a thing to be avoided whenever possible; but the time is long past when statistical methods should have been accepted and earnestly put to use by the engineer.

The objects of this paper are three: First, to present a correlation table in a new light; second, to offer graphical methods which may be used for other but similar purposes; and, third, to provide a short method for obtaining means, standard deviations, and possibly correlation coefficients for those to whom computing machines are not available, but who are experienced draftsmen.

Methods

The methods described apply to data consisting of two variables which bear a relationship to each other. The problem involved is to determine the degree of this relationship as measured by the correlation coefficient.

The correlation table shown in Fig. 2 was made from data consisting of the 7-day tensile ratios and the 28-day tensile ratios for mortars using 147 sands. For any sand the ratio is found by dividing its strength by that for

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* Published in January, 1929, *Proceedings*.

† Asst. Prof., Civ. Eng., Univ. of Maine, Orono, Me.

‡ For a solution of correlation tables by usual methods and other statistical methods, see Yule's "Introduction to the Theory of Statistics," p. 381, J. B. Lippincott Co., Philadelphia, Pa.

Ottawa sand made with the same cement at the same time. These ratios were multiplied by 100, thus expressing them as percentages.*

The 7-day ratios are plotted on the horizontal axis and the 28-day ratios on the vertical axis. For convenience, the arrays are numbered consecutively from 1 to 12, beginning in the upper left-hand corner. For the present purpose the numbers in the centers of the squares must be assumed to represent units of force acting at the center of the square. The numbers at the bottom of the diagram would then represent the units of force acting vertically in the respective columns, and the numbers at the right side represent units of force acting horizontally in the respective rows.

The mean value of X and the mean value of Y must first be obtained. This is easily accomplished by applying the method for finding the resultant of any system of forces.†

The next constants sought will be the standard deviations of the X and Y-values. The standard deviation of Y ($S.\ D.\ y$) is found by obtaining the summation of the products of the units of force and the squares of the distances from the mean Y-line, dividing by the number of units in the table, and extracting the square root. To the engineer this means $\sqrt{\frac{\Sigma\,(A\,y^2)}{\Sigma\,A}}$, in which, A may be replaced by n where n is the number in any given square, and y is the distance from the center of the row containing the square to the Mean-Y line. The standard deviation of X ($S.\ D.\ x$) is obtained in a similar manner.

To find the value of $\Sigma\,(A\,x^2)$, let $O\,X$ be the X-axis and $O\,Y$ the Y-axis (Fig. 1) and assume that the area, A, is able to exert a force of A-units parallel to either axis.

Lay off the vector, 0-1, to represent A to some given scale. Choose a point, p, any perpendicular distance, such as H_x, from 0-1 and draw 0-p and 1-p. Draw a line through A parallel to $O\,Y$. From any point, m, on this line, draw a-m parallel to 1-p, and then m-b parallel to 0-p. Now, chose a pole, p', any perpendicular distance, such as H_y, from a-b and draw a-p' and b-p'. Draw f-n parallel to a-p' and n-e parallel to b-p'. Then,

$$A\,x^2 = (e\text{-}f) \times H_y \times H_x \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The proof follows: Triangle $p\,0\,1$ is similar to Triangle $a\,m\,b$, and Triangle $p'\,a\,b$ is similar to Triangle $f\,n\,e$. In the first pair of triangles:

$$0\text{-}1 : H_x = a\text{-}b : x$$

or,

$$(0\text{-}1) \times x = H_x \times (a\text{-}b)$$

But,

$$(0\text{-}1) \times x = A \times x = H_x \times (a\text{-}b)$$

$$a\text{-}b = \frac{A \times x}{H_x}$$

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* For information concerning similar data, see, *Bulletin No. 10,* Maine Technology Experiment Station.

† See "Graphical Analysis," by William S. Wolfe, McGraw-Hill Book Co.

In the other triangles,

$$a\text{-}b : H_y = e\text{-}f : x$$

$$(a\text{-}b) \times x = H_y \times (e\text{-}f)$$

Substituting the value of $a\text{-}b$,

$$\frac{A \times x \times x}{H_x} = H_y \times (e\text{-}f)$$

or,

$$A\,x^2 = (e\text{-}f) \times H_x \times H_y \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

This same method will be applied to each of the squares in the diagram and the result, $\Sigma\,(A\,x^2)$, obtained by multiplying one ordinate by H_x and H_y.

The value of $A \times x \times y$ is next required. Proceed as before to the location of p' and the drawing of $a\text{-}p'$ and $b\text{-}p'$. Then find the intercept between $a\text{-}p'$ and $b\text{-}p'$ on a line parallel to $O\,Y$ at a distance of y from p'. $A \times x \times y$ is then equal to $(c\text{-}d) \times H_x \times H_y$.

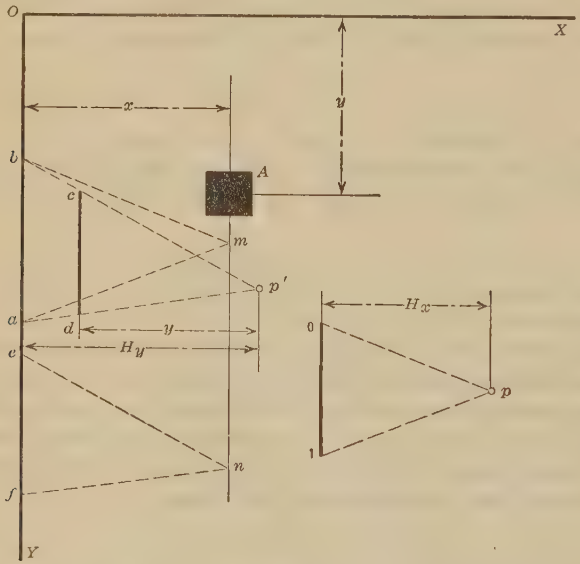

FIG. 1.

The proof follows: As before, Triangles $a\,m\,b$ and $p\,0\,1$ are similar. Then,

$$a\text{-}b : x = 0\text{-}1 : H_x$$

or,

$$x \times (0\text{-}1) = (a\text{-}b) \times H_x$$

But,

$$x \times 0\text{-}1 = A \times x = (a\text{-}b) \times H_x$$

Since Triangles $p'\,a\,b$ and $p'\,d\,c$ are similar,

$$a\text{-}b : c\text{-}d = H_y : y$$

therefore,

$$(a\text{-}b) \times y = (c\text{-}d) \times H_y, \text{ or } y = \frac{(c\text{-}d) \times H_y}{a\text{-}b}$$

As shown, $A \times x = (a\text{-}b) \times H_x$. Then,

$$A \times x \times y = (c\text{-}d) \times H_x \times H_y \ldots\ldots\ldots\ldots\ldots\ldots(3)$$

In the correlation table to be solved (Fig. 2), the ordinate for $\Sigma\ (n\,x\,y)$ for each row may be found at once and these may be added, provided the H_x and H_y remain the same for all the rows. Then, $\Sigma\, n\, x\, y$ for the the whole diagram may be found by multiplying this total ordinate by H_x and H_y, all values being measured to the proper scale.

One of the most difficult features in applying these methods to a larger problem is the choice of scales, two of which are used, namely, the scale of percentage and the scale of numbers. On Figs. 2, 4, 5, 7, 8, 10, and 11, all measurements are made to the scale of percentage, that is, a given length represents a certain percentage. In Figs. 3, 6, and 9, all measurements are made to the scale of numbers and a given length represents a number of units. Table 1 will be helpful in following the demonstration.

TABLE 1.

Symbol.	Scale.	Magnitude.
H_y	Number.	100
H_x	"	100
H_{xy}	"	40
H_{y-1}	Percentage	20
H_{x-1}	"	20
H_{xy-1}	"	20
$S.\ D.\ y$	"	20.0
$S.\ D.\ x$	"	20.1

SOLUTION

These methods will now be applied to the correlation table shown in Fig. 2. The graphical construction necessary to solve the table is given in Figs. 3 to 11, inclusive. These diagrams are numbered consecutively in the order of their use,* the over-lapping one of another (especially the various Figs. 10 (*a*), 10 (*b*), etc., on Fig. 2) is unavoidable and is essential to the graphical accuracy.

In Fig. 3, lay off 0-1 representing to the chosen scale the number of units in the first row. There are no units in the second row. Lay off 1-2 representing the number of units in the third row and proceed similarly for all the rows getting the line, 0-11. Choose any point, P_1, some convenient distance, such as H_y, from 0-11 and draw the rays, P_1-0, P_1-1, etc. Next, draw horizontal lines through the centers of all the rows as shown on Figs. 4 and 5. On Fig. 4, draw String P_1-0 parallel to Ray P_1-0 of Fig. 3, to meet the horizontal line through the center of the row at *a*. Draw String P_1-1 parallel to Ray P_1-1 to get Line *a*-*b*. Thus continue to get the line, *a*-*b*-*c* * * * *m*. Extend P_1-0, the first string, to meet P_1-11, the last string at 0. This point is on the Mean-*Y* line. This line now becomes the *X*-axis and values of *y* and moments derived therefrom refer to it. Next, extend the string, P_1-1, to meet the Mean-*Y* line at 1 and so for all the strings to get the points, 0-1-2- * * * 11 on the Mean-*Y* line. Choose the point, p_1, some convenient distance, H_{y-1},

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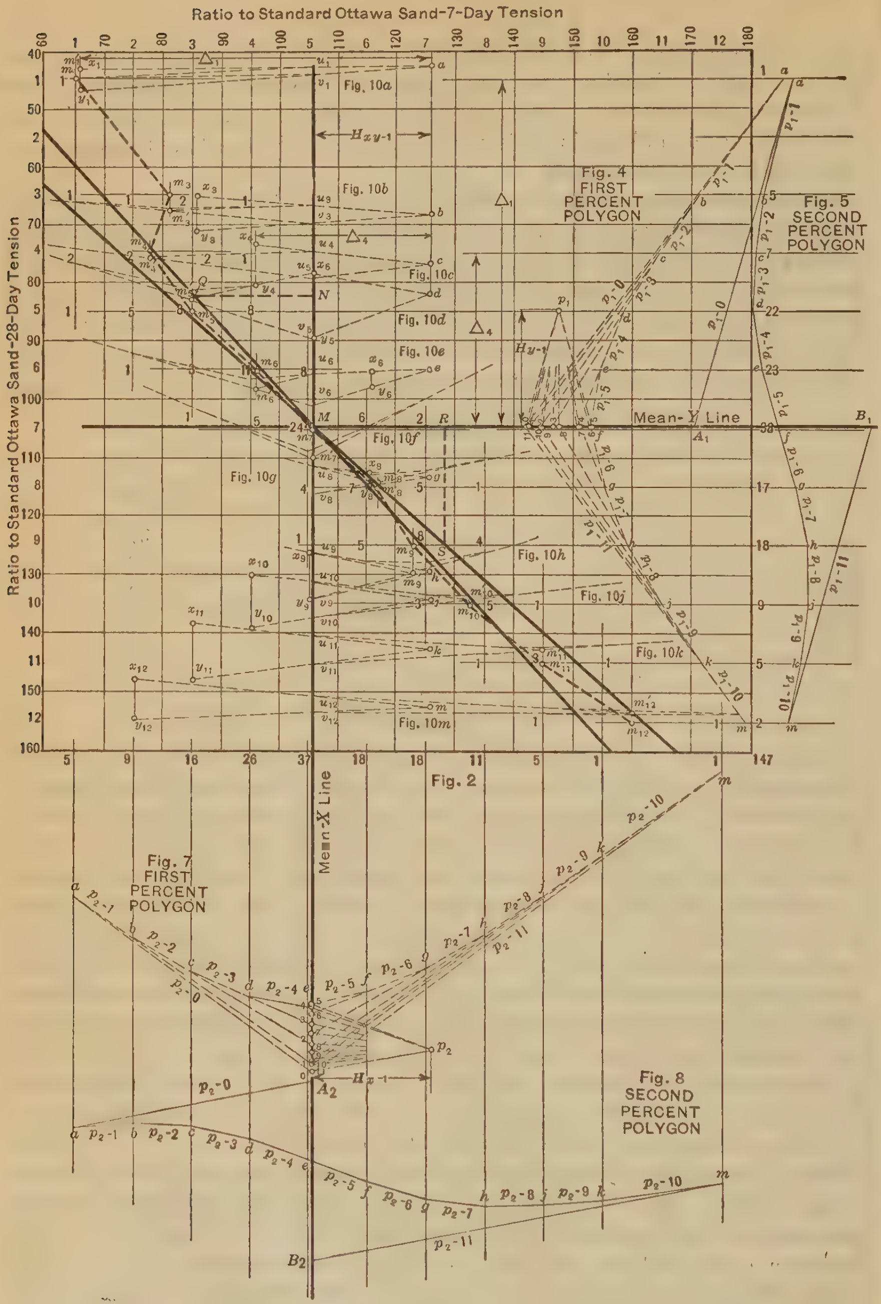
* In actual practice Figs. 3 to 11 would best be drawn on a single sheet for graphical comparison; for convenience, they are separated here into two groups, which, however, should be considered jointly.


Ratio to Standard Ottawa Sand-7-Day Tension
Ratio to Standard Ottawa Sand-28-Day Tension
60 70 80 90 100 110 120 130 140 150 160 170 180
1 2 3 4 5 6 7 8 9 10 11 12
40 50 60 70 80 90 100 110 120 130 140 150 160
Fig. 10a
Fig. 10b
Fig. 10c
Fig. 10d
Fig. 10e
Fig. 10f
Fig. 10g
Fig. 10h
Fig. 10j
Fig. 10k
Fig. 10m
H_{xy}-1
H_y-1
H_x-1
Δ₁
Δ₄
Fig. 4
FIRST
PERCENT
POLYGON
Fig. 5
SECOND
PERCENT
POLYGON
Mean-Y Line
A_1
B_1
M
N
Q
R
S
Fig. 2
5 9 16 26 37 18 18 11 5 1 1 147
Mean-X Line
Fig. 7
FIRST
PERCENT
POLYGON
Fig. 8
SECOND
PERCENT
POLYGON
A_2
B_2
p_1
p_2
p_1-0 p_1-1 p_1-2 p_1-3 p_1-4 p_1-5 p_1-6 p_1-7 p_1-8 p_1-9 p_1-10 p_1-11
p_2-0 p_2-1 p_2-2 p_2-3 p_2-4 p_2-5 p_2-6 p_2-7 p_2-8 p_2-9 p_2-10 p_2-11
a b c d e f g h j k m
1 5 7 22 23 38 17 18 9 5 2

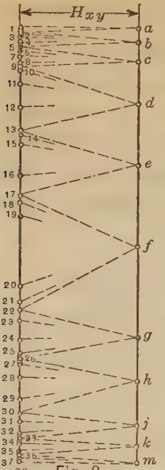

H_{xy}
a
b
c
d
e
f
g
h
j
k
m
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
37
38

Fig. 9
NUMBER
POLYGON
Scale in Units
0 10 20

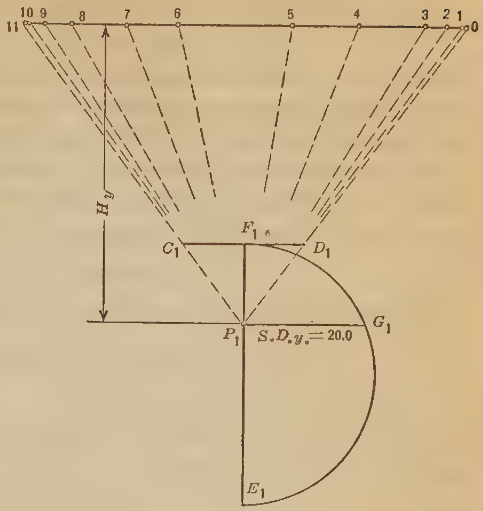

10
9
8
7
6
5
4
3
2
1
0
11
H_y
F_1
C_1
D_1
P_1
G_1
S.D.y. = 20.0
E_1

Fig. 3
NUMBER
POLYGON
Scale in Units
0 10 20

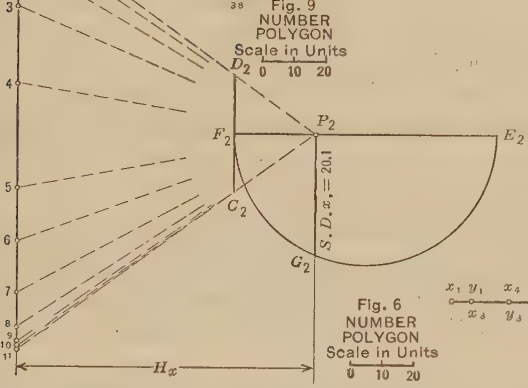

0
1
2
3
4
5
6
7
8
9
10
11
D_2
P_2
F_2
E_2
C_2
G_2
S.D.x. = 20.1
H_x

Fig. 6
NUMBER
POLYGON
Scale in Units
0 10 20

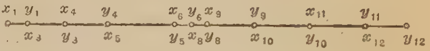

x_1
y_1
x_4
y_4
x_6
y_6
x_9
y_9
x_{11}
y_{11}
x_3
y_3
x_5
y_5
x_8
y_8
x_{10}
y_{10}
x_{12}
y_{12}

Fig. 11

from this line and draw Rays p_1-0, p_1-1, etc. In Fig. 5 draw String p_1-0 parallel to Ray p_1-0 of Fig. 4 and get Line a-A_1; then, draw a string parallel to p_1- and get a-b. Continue this procedure until m-B_1 is obtained. The value, A_1-B_1, is used to obtain the standard deviation.

In Fig. 3, lay off C_1-D_1 parallel to 0-11 and equal to H_{y-1} in Fig. 4. Through P_1 draw a line, F_1-E_1, perpendicular to C_1-D_1, so that P_1-E_1 is equal to A_1-B_1 in Fig. 5. With F_1-E_1 as a diameter, draw the semi-circle, F_1-G_1-E_1. Then, through P_1 draw P_1-G_1 perpendicular to F_1-E_1. This line, P_1-G_1, gives the standard deviation of Y when measured to the Y-scale on the diagram.

The proof follows: By construction, Triangle 0-11-P_1 is similar to Triangle P_1-C_1-D_1. Then,

$$0\text{-}11 : H_y = C_1\text{-}D_1 : F_1\text{-}P_1.$$

But 0-11 = number of individuals in the table = N; and C_1-D_1 = H_{y-1}. Therefore,

$$N : H_y = H_{y-1} : F_1\text{-}P_1$$

or,

$$N\ (F_1\text{-}P_1) = H_y \times H_{y-1} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

From Equation (1) and the theorem derived therefrom,

$$\Sigma n y^2 = (A_1\text{-}B_1)\ H_y\ (H_{y-1})$$

in which, n is the number in any square and y is the distance from the Mean-Y line to the center of this square. This is seen to be true, as the ordinate, A_1-B_1, may be divided into a large number of smaller ordinates exactly comparable to the ordinate e-f in Equation (1). This division may be made by extending the strings in Fig. 5 to meet the Mean-Y line. By construction, P_1-E_1 = A_1-B_1; therefore,

$$\Sigma\ (n y^2) = (P_1\text{-}E_1)\ H_y\ H_{y-1}$$

Substituting the value of $H_y\ H_{y-1}$ of Equation (4),

$$\Sigma\ (n y^2) = (P_1\text{-}E_1)\ N\ (F_1\text{-}P_1)$$

or,

$$\frac{\Sigma\ (n\ y^2)}{N} = (P_1\text{-}E_1)\ (F_1\text{-}P_1)$$

But,

$$S.\ D.\ y = \sqrt{\frac{\Sigma\ (n\ y^2)}{N}} = \sqrt{(P_1\text{-}E_1)\ (F_1\text{-}P_1)}$$

From plane geometry, P_1-G_1 is a mean proportional between P_1-E_1 and F_1-P_1. Therefore,

$$P_1\text{-}G_1 = \sqrt{(P_1\text{-}E_1)\ (F_1\text{-}P_1)} = S.\ D.\ y$$

The mean value of X and the standard deviation of X may be obtained in the same manner using the columns instead of the rows. This construction is shown in Figs. 6, 7, and 8. Values of x refer to the Mean-X line as an axis and the intersection of the Mean-X and Mean-Y lines, marked M, is the origin.

The value, $\Sigma\ n\ x\ y$, is known as the product of inertia, the observations being considered as having weight. This may be simplified by finding $\Sigma\ n\ x$ for each row (Fig. 2) and multiplying it by the y for this row indicated by Δ_1 for the first row and Δ_4 for the fourth row. The y for other rows is not shown. In Fig. 9, lay off 0-1 to represent the number of units in the first square of the first row. There being no other units in this row, this part of Fig. 9 is now complete. Choose any point, a (Fig. 9), some convenient distance from 0-1, as H_{xy}, and draw Rays a-0 and a-1. In Fig. 10 (a), assume m_1' anywhere on the center line of the first square and draw String m_1'-v_1 parallel to Ray a-0 of Fig. 9 and String m_1'-u_1 parallel to Ray a-1. In general, u_1 and v_1 are the points on the Mean-X line where it is intercepted by the first and last strings, and m_2' is the intersection of the first and last strings, thus locating the mean value of the observations in the row under consideration. For purposes to be explained, m_1 is located directly under m_1', and on the center of the row. This line, u_1-v_1, when multiplied by H_{xy}, using proper scales, gives the moment of this unit about the Mean-X line, or $\Sigma\ n\ x$, for this row.

It is now desired to obtain $\Sigma\ n\ x\ y$ or $\Sigma\ n\ x\ \Delta_1$. $\Sigma\ n\ x$ may be represented by u_1-v_1. In Fig. 10 (a) choose some point, a, any convenient distance, such as H_{xy-1}, from the Mean-X line. Then, a-u_1-v_1 becomes a force polygon in the ordinary meaning of the word. By modifying somewhat the theorem for simple moment, an intercept between strings drawn parallel to Rays a-u_1 and a-v_1, on a vertical line a distance, Δ_1, from a, will give the result sought. Extend a-u_1 and a-v_1 to get the points, x_1 and y_1, these points being located so that x_1-y_1 is parallel to the Y-axis and so that the perpendicular distance from a to x_1-y_1 (Fig. 10 (a)), equals Δ_1.

From the theorem for simple moment:

$$(u_1\text{-}v_1)\ H_{xy} = \Sigma\, n\, x$$

$$(x_1\text{-}y_1)\ H_{xy_{-1}} = (u_1\text{-}v_1)\ \Delta_1$$

By construction, $y = \Delta_1$; therefore,

$$\Sigma\ (n\, x\, y) \text{ for the first row} = (x_1\text{-}y_1)\ (H_{xy})\ (H_{xy_{-1}})$$

Although the procedure for all the rows is the same, one with a larger number of units will be used to demonstrate. Consider the fourth. In Fig. 9, the distance, 5-6, represents to scale the two units in the first square of this row; 6-7, the two units in the second square; 7-8, the two units in the third square; and 8-9, the single unit in the fourth square.

The point, c, is chosen so that its distance from the number line, 0-38, is the same as that of a, or H_{xy}. All these points, a, b, * * * m, must be so located. Now, draw Rays c-5, c-6, c-7, c-8, and c-9. In Fig. 10 (c) start preferably so that m_4' will fall in Row 4 and draw strings parallel to the corresponding rays in Fig. 9 as for any space polygon. The first and last strings produced will intersect at m_4', locating the mean of the observations in this row. This point projected upward to the center of the row gives the point, m_4, which corresponds to m_1 for Row 1. The first and last strings produced to meet the Mean-X line gives the points, u_4 and v_4. Through c, Fig. 10 (c), located as was a in Fig. 10 (a), draw c-u_4 and c-v_4 and extend these lines to get the points, x_4 and y_4, these points being so located that Line x_4-y_4 is parallel to the Y-axis and a perpendicular distance from c equal to Δ_4.

This construction is similar to that for the first row, hence the intercept, x_4-y_4, when measured to the proper scale and multiplied by H_{xy} and $H_{xy_{-1}}$ will give $\Sigma\, n\, x\, y$ for this row. Since H_{xy} and $H_{xy_{-1}}$ are made constant for all the rows, the ordinates may be added and the multiplication made in one operation. This is shown in Fig. 11. It should be noted that $\Sigma\, n\, x\, y$ for the seventh row is zero, since the first and last strings intersect on the Mean-X line.

With $\Sigma\, n\, x\, y$, $S.\ D.\ x$, $S.\ D.\ y$, and N known, r, may be found. In this case, from Fig. 11,

$$\Sigma\, x\, y = (X_1\text{-}y_{12})\ (H_{xy_{-1}})\ (H_{xy}) = 66.4 \times 20 \times 40 = 53\,120$$

Then,

$$r = \frac{\Sigma\, x\, y}{N \times S.\ D.\ x \times S.\ D.\ y} = \frac{53\,120}{147 \times 20.1 \times 20} = 0.900$$

This denotes a high relationship and shows that, in general, those sands having a high 7-day tensile ratio will have a high 28-day tensile ratio. This conclusion could have been reached by inspection, but the absolute value of the relationship could not be found for further use or for comparison with other results.

The regression lines are readily drawn by graphical methods. Points m_1, m_3, etc., have been located on the mean value of the rows. A broken line joining these points is called the "raw regression line", and the straight line that best fits this broken line is called the "mean regression line" (see Fig. 12). This line shows how x regresses on y. A given set of observations having identical y-values would not have identical x-values, but there would be considerable range or spread. For example, referring to Fig. 2, it may be seen that those briquettes having 28-day tensile ratios between 70 and 80% had 7-day tensile ratios ranging from 60 to 100 per cent. It would be expected, however, that the mean of the 7-day tensile ratios for this row would be about 80%, as indicated by the mean regression line.

If the mean of the x-values in any row had been plotted instead of the actual values, these points would have fallen nearly on a straight line, and the facts sought for might have been found approximately without the aid of correlation. Unfortunately, this has been done by many investigators, but the procedure is wrong, for the cause of this spread may be the solution of the problem and as soon as averaging begins the cause is buried never to be uncovered. The raw regression line must be plotted to determine the linearity of the variation, but a study should be made of the entire data, not of a few average results. The mean regression line of x on y may be drawn as follows:

Designate the point of intersection of the mean lines as M. From M lay off a distance, M-N, on the Mean-X line equal to $\frac{S.\,D.\,y}{r}$. At N erect a perpendicular, N-Q, equal to $S.\,D.\,x$. Q-M is the mean regression line sought. This is seen to be correct from the regression equation,

$$x = \text{Mean-}X - r\,\frac{S.\,D.\,x}{S.\,D.\,y}\,(\text{Mean-}Y - y)$$

This may be stated as,

$$x = \text{Mean-}X - S.\,D.\,x\,.\frac{(\text{Mean-}Y - y)}{\frac{S.\,D.\,y}{r}}$$

Now, when $(\text{Mean-}Y - y) = \frac{S.\,D.\,y}{r}$, or $y = \text{Mean-}Y - \frac{S.\,D.\,y}{r}$,

$$x = \text{Mean-}X - S.\,D.\,x$$

This point (x, y) has thus been located. The other raw regression line, that of y on x, must be located by finding the centers of the columns in the same manner as the centers of the rows. To locate the corresponding mean regression line, lay off on the Mean-Y line, a distance, M-$R = \frac{S.\,D.\,x}{r}$. Then, lay off an ordinate, R-S, perpendicular to M-R and equal to $S.\,D.\,y$. This locates the desired regression line, M-S. Predictions may be made from these lines.

While this method of studying widely variable data will appeal to only a few people, it should be remembered that graphical methods have not gone through the process of evolution experienced by arithmetical methods. Furthermore, good draftsmanship is required to make the method a success. While the solution of r will increase in difficulty as the number of units in the table increases, principally because of the determination for $\Sigma\, n\, x\, y$, the solution for the standard deviations is affected but little, and the method of plotting the mean regression lines is not affected at all.

The accuracy of the results will depend to a large extent on the scales used. However, careful work and ordinary size drawings should give results accurate to the third decimal place. Where pictorial results are required, these methods should prove very satisfactory.

For purposes of comparison the arithmetical solution is shown on Fig. 12. To the right of the diagram, which is similar to that shown in Fig. 2, are various numerical values. In Column (1), for example, will be found the number of observations, N, falling in each row. In the graphical solution, the Mean-X line was used for a Y-axis; hence, $\Sigma\, n\, x$ for the table was zero. This is the ideal method, but for a mathematical solution it would necessitate replotting the table or at least a tedious process of getting variations from the mean with which to obtain the required summations. It is much easier to assume an origin and, after the mean values are found, change the results by the use of a transfer formula.

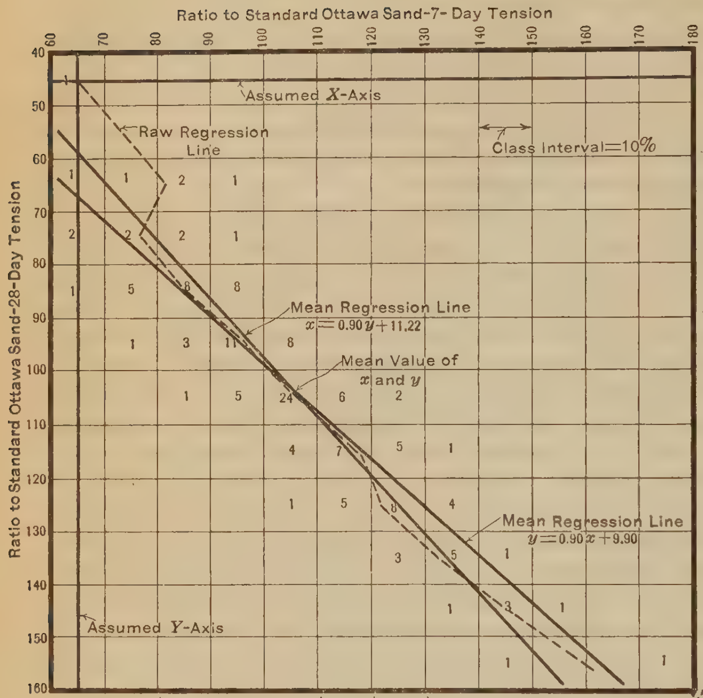
For convenience, the Y-axis should be chosen so that x-values will be as small as possible, but not negative. It will be assumed to pass through the center of the first column as indicated on Fig. 12. The same results are obtained with much less work if x and y are expressed in terms of class interval, or, in this case, units of ten, rather than in percentage; that is, the difference between the center of the first column and the center of the second would normally be ten, but is used as unity. For the present the numbers along the axes have lost their meaning. Consider the fourth row for illustration. Taking the observations in each column in order from left to right, $\Sigma\, n\, x$ for the row becomes, $(2 \times 0) + (2 \times 1) + (2 \times 2) + (1 \times 3) = 9$, the result found in Column (2), Fig. 12. Values in Column (3) are the means of the observed values in the respective rows and correspond to the points, m, in Fig. 2.

At the bottom of the table are seven rows which correspond to the seven columns at the right, except that the x's and y's are interchanged. The results are the same, except in Row 2 where the mean values for the columns are found instead of the mean values of the rows. The summations are now complete.

The standard deviations of x and y, also the correlation coefficient, are found as indicated on Fig. 12. These standard deviations and the means are, however, in terms of class-interval rather than percentage. The true $S.\,D.$ is equal to the $S.\,D.$ as found, multiplied by the class-interval. Therefore:

$$S.\,D.\,x = 2.008 \times 10 = 20.08\%$$

$$S.\,D.\,y = 2.002 \times 10 = 20.03\%$$



1	2	3	4	5	6	7
N'	$\Sigma n x$	$\frac{\Sigma n x}{N'}$	$\Sigma n x y$	$\Sigma n x^2$	$\Sigma n y$	
1	0	0	0	0	0	
5	8	1.6	16	18	10	
7	9	1.3	27	19	21	
22	45	2.0	180	109	88	
23	72	3.1	360	240	115	
38	155	4.1	930	655	228	
17	88	5.2	616	468	119	
18	105	5.8	840	625	144	
9	61	6.8	549	417	81	
5	40	8.0	400	322	50	
2	19	9.5	209	185	22	24
147	602	.....	4 127	3 058	878	

1 N'	5	9	16	26	37	18	18	11	5	1	......	1	147
2 $\Sigma n y$	12	33	63	122	220	125	138	94	50	10	......	11	878
3 $\frac{\Sigma n y}{N'}$	2.4	3.7	3.9	4.7	5.9	6.9	7.7	8.5	10.0	10.0	......	11.0	.....
4 $\Sigma n x y$	0	33	126	366	880	625	828	658	400	90	......	121	4 127
5 $\Sigma n y^2$	38	127	265	596	1 324	879	1 072	810	502	100	......	121	5 834
6 $\Sigma n x$	0	9	32	78	148	90	108	77	40	9	......	11	602
7 $\Sigma n x^2$	0	9	64	234	592	450	648	539	320	81	......	121	3 058

N = No. of samples = 147

$$S.D.x=\sqrt{\frac{\Sigma n x^2}{N}-\left(\frac{\Sigma n x}{N}\right)}$$

$$=\sqrt{\frac{3\ 058}{147}-\left(\frac{602}{147}\right)^2}=2.00$$

$$S.D\ y=\sqrt{\frac{\Sigma n y^2}{N}-\left(\frac{\Sigma n y}{N}\right)}$$

$$=\sqrt{\frac{5\ 834}{147}-\left(\frac{878}{147}\right)^2}=2.00$$

$$r=\frac{\frac{\Sigma n x y}{N}-\frac{\Sigma n x}{N}\cdot\frac{\Sigma n y}{N}}{S.\,D.\,x\times S.\,D.\,y}$$

$= 0.899$

Probable error of r

$$=\frac{0.6745\,(1-r^2)}{\sqrt{N}}=0.011$$

FIG. 12.—ARITHMETICAL SOLUTION OF A CORRELATION TABLE.

The true mean is equal to the mean as found, multiplied by the class-interval, plus the interval between the assumed axis and the true axis passing through 0, 0 (outside the limits of Fig. 12):

$$\text{Mean-}X = (4.095 \times 10) + 65 = 105.95\%$$

$$\text{Mean-}Y = (5.973 \times 10) + 45 = 104.75\%$$

These true values must be used to obtain the mean regression lines.

The raw regression line of x on y is drawn by plotting the values in Column (3), Fig. 12, in the respective rows and connecting the points. The raw regression line of y on x is drawn by plotting the values in Row 3 in the respective columns and connecting the points.

The mean regression line of x on y is given by:

$$x = \text{Mean-}X - r\frac{S.\ D.\ x}{S.\ D.\ y}(\text{Mean-}Y - y) \text{ ....} (5)$$

referred to the origin at 0, 0.

If this be referred to the intersection of the mean lines as an origin, it becomes $x = r\frac{S.\ D.\ x}{S.\ D.\ y}y$, and if the $S.\ D.\ x = S.\ D.\ y$ as it does, practically, in this case,

$$x = r\,y \text{ ....} (6)$$

Here is seen a tangible meaning for the value of r.* It is the slope of a line, for should the standard deviations be unequal this obstacle may be overcome by arranging scales so that, graphically, the two standard deviations will be represented by the same length of line.

The other mean regression line is, in the first form,

$$y = \text{Mean-}Y - r\frac{S.\ D.\ y}{S.\ D.\ x}(\text{Mean-}X - x) \text{ ....} (7)$$

or, in the simple form,

$$y = r\,x \text{ ....} (8)$$

For the table (Fig. 12) as made, the first form should be used, giving the following equations:

$$x = 0.90\,y + 11.59 \text{ ....} (5a)$$

$$y = 0.90\,x + 9.74 \text{ ....} (7a)$$

Apparently, it is common practice among engineers to average results and consider as their data what is actually only one raw regression line. It may be seen that the equation derived will depend somewhat on how the data are averaged. The means of the rows give different results from the means of the columns. If it is desired to predict 28-day ratios from 7-day ratios, then Equation (7*a*) must be used; but if it is desired to predict 7-day ratios from 28-day ratios, then Equation (5*a*) must be used. Present methods probably would place the line between these two, and the results would be in error accordingly. Under any circumstances it would seem that the usual practice—plotting the data, drawing the best looking line the equation of which may be easily obtained, and calling this good enough—should be abandoned.

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* This relationship was first stated by Francis Galton about 1880.

The mathematical solution is now complete. About four hours should be required to solve the table completely by either method.

Conclusion

The first purpose of this paper was to present the correlation table in a new light. It has been shown that the term called "standard deviation" may be obtained by those same methods by which for many years engineers have evaluated the radius of gyration of any shape. The $\Sigma n x y$ is known to the Engineering Profession as the product of inertia, and means are located as is the center of gravity. With the regression lines plotted, the table may be used very readily to predict one quantity from a known value of a related quantity.

The second purpose of the paper was to present graphical methods which might be used for similar purposes, such as to find centers of gravity, moments of inertia, and products of inertia of irregular areas which are not handled easily by any other method.*

The third purpose of the paper was to offer a short method to obtain means and standard deviations. Where the table must be drawn, and where frequency curves and regression lines are required, it is believed that this method will prove efficient, as the extra work for completing the solution will not be great.

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* These methods are treated in the text referred to in the foot-note on p. 937, with the exception of the product of inertia.

DISCUSSION

WALTER H. DUNLAP,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is a well written exposition of the application of graphical methods to a statistical problem, but the writer thinks that while the graphical solution has certain advantages, they are not great enough to make up for the additional time required.

The writer believes that Professor Evans has over-estimated the time required for the arithmetical method. Dr. Frank McG. Phillips, of the U. S. Bureau of Education, has solved this problem by arithmetic in 20 min., using one and one-half sheets of letter-size paper. While it is not fair to assume that the time required for an expert is normally to be expected in every case, the writer believes that 4 hours is too long.

The advantage of the graphical solution is in the understanding and grasp derived of the problem of correlation while learning about this method of analysis; but after having acquired a fairly good understanding of the method, the regular run of such problems may best be solved by the arithmetical method with the aid of a computing machine. In college the graphical solution is used in the design of a plate girder or a Pratt truss, but the practicing bridge engineer uses the arithmetic method for such a standardized problem. The correlation problem presented by Professor Evans is in much the same category. It is a job to be turned over to an assistant or a computer the same as the computation of the closure and area of a survey of a boundary would be. In fact, some of the computing-machine manufacturers have perfected and are selling a correlation computing machine. However, it is designed for the computation of correlations between time series rather than frequency distributions.

In Professor Evans' problem, the lines of regression are straight and the coefficient of correlation is unusually high. In a problem in which the correlation table clearly indicates a curved line of regression, the writer wonders how the graphical solution would be modified and whether it would show an advantage over the arithmetic solution. For example, suppose the raw regression line (Fig. 12) in the problem had shown a much more pronounced tendency to curve up at the ends than it does, thus indicating the necessity of fitting a regression line of the form, $y = a + b\,x + c\,x^2$, to the data, would the solution of the problem by the graphical method be more advantageous than that by the arithmetic solution?

Three of four places of decimals are quite sufficient for the problem presented, but for partial correlation, additional places are necessary as additional factors are added. For example, if it were desired to analyze the recorded results of many tests of strength of concrete for the purpose of estimating the effect produced by various proportions of cement, sand, crushed stone, water, etc., the methods of partial correlation are indicated as applicable. Partial correlation provides a means of estimating the effect produced by each individual component cause on the assumption that all other causes are held constant.

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It is better to take the assumed origin as near as possible to the center of gravity of the table than to place it in one corner for the sake of avoiding negative values. The position of the center of gravity can usually be approximated by inspection; in the author's example it is in the block containing a "frequency" of 24 samples. The advantage is the same as that of assuming the meridian near the middle of a boundary in computing the area; it results in smaller figures with which to work. The correlation table (Fig. 12) in the problem is a comparatively small one. In a table having 18 or 20 or perhaps 25 classes each way, with the total number of cases, N, as much as 1 000, or more, the arithmetic is much simpler if the assumed origin is taken near the center of gravity of the table. One can multiply in his head by 12, but if the multiplier becomes 18 or 23, for example, it is easier to use paper and pencil. By assuming the origin near the center of gravity, virtually all the work down to the computation of the standard deviations can be done in the head, because the multipliers will seldom be greater than 12. For example, in Fig. 12, it will be noted that in the fourth row, $N = 22$, $\Sigma\, n\, y = 22 \times 4 = 88$, and $\Sigma\, n\, y^2 = \Sigma\, n\, y \times y = 88 \times 4 = 352$.

In addition to the text by Yule, cited by Professor Evans (which may be regarded as standard), the writer recommends* a text by Frederick C. Mills, Associate Professor of Business Statistics at Columbia University.

ROBERT C. STRACHAN,† M. AM. SOC. C. E. (by letter).—The author lays more stress on methods of obtaining "standard deviation", "coefficient of correlation", and "mean regression line", than on the question, "Why a correlation table?"

As the answer is by no means self-evident, it seems proper to state that—in connection with the theory of probabilities, with its voluminous literature concerning frequency curves, probability curves, frequency surfaces, normal law of error, and kindred subjects—the correlation table or diagram has been developed for the purpose of exhibiting two observed properties or qualities pertaining in different degree to a group of objects, in such a way that the probable influence of either property on the other, may be made apparent.

Thus, the regression lines, Equations (5*a*) and (7*a*), or their loci in Fig. 12, indicate that if a mortar similar to the tested group has a strength ratio of 80 at 7 days, it should have a ratio of 81.9 at 28 days. On the other hand, for a mortar showing a ratio of 90 at 28 days, the probable ratio is 92.2 at 7 days. These are, of course, expectations founded on the averages represented by the regression lines.

The data chosen by the author to illustrate his method are of a type occurring frequently in engineering work. For example, the effect of temperature on electrical resistance, the influence of percentage of constituent metals on the strength of an alloy, or that of wire-drawing on the elastic modulus of steel, might be studied effectively by similar means.

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* "Statistical Methods Applied to Economics and Business", by Frederick C. Mills, Henry Holt & Co., New York, N. Y.

† Cons. Engr., New York, N. Y.

The author's graphical derivation of the elements essential to the building of the diagram is ingenious, and gives evidence of much study. It also tends to prove that most problems, however complex, may be made to yield to graphics. The desideratum, however, is clearness combined with a simplicity commensurate with the conditions of the problem. Clearness and simplicity appear to the writer to characterize the arithmetical summarization of Fig. 12, rather than Figs. 4 to 10, inclusive.

The author estimates that 4 hours should be sufficient for the complete solution of this particular table by either procedure; so that no superiority is claimed for the graphical method on the score of speed. The writer's impression is that if a 4-hour record for this work exists, it must have been made by an exceedingly clever draftsman.

S. L. MOYER,* ESQ. (by letter).—In translating the probable error, *r*, into a spread between two bounds of a zone of uncertainty, Professor Evans attaches a graphic meaning to this elusive index that is most interesting. While the capacity of mathematics to express the degree of credibility may always be subject to question, yet the divergence between the mean regression lines of X on Y and of Y on X in Fig. 12 does furnish a portrait of the relative accuracy of the different parts of the graph of conclusions, reflecting the wealth or scarcity of the data supporting it.

The procedure involves the method of least squares as applied by Professor Karl Pearson, and his collaborators to the scatter of observations along the two axes of X and Y. As a practical test of the validity of these methods, Professor Pearson's favorite experiment was to examine the scatter of 1 000 shots fired at a line on a target, and these examinations constitute the only positive proof of the departure characteristics presumed to exist. Under this method of interpretation, the trend or true value at which the shots are conceived to be aimed, is missing; and the problem is to discover this line without giving an unwarranted weight to a few scattering values.

The method of least squares was used by Karl Friedrich Gauss as early as 1795, and as later shown by him, it depends for its validity on the Gaussian Law of Error. This law results from a generalization of the binomial theorem known as the multi-nomial theorem. It is, in effect, an indefinite extension of the algebra of a game of dice, in which the numbers or quantities accidentally united, are conceived to add to produce the phenomena or score. This whole mathematical structure, therefore, rests on the premise that the haphazard values of the elemental causes unite by addition to produce the scatter effect.

It seems reasonable to believe that the human, mechanical, and other variations incident to gunfire are most likely to unite, by an addition of the haphazard contributions from these sources, in influencing the scatter of shots on the target. In fact, there seems to be very little question concerning the applicability of the Gaussian law to the reconcilement of the ordinary human and mechanical errors of observation, all of which may be considered as exerting an additive influence on the haphazard combination of causes. In the case of natural phenomena, however, a great number of the known results are multiples of the causes, rather than summations, and it seems worth while to bear this fact in mind as a check on any unwise assumption of accuracy which may be ascribed to a reconcilement of this kind.

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* Civ. Engr., Montevideo, Minn.

The various increments of the chance in the haphazard throw of one die, may be symbolized by the successive terms in the following expression: $x^1 + x^2 + x^3 + x^4 + x^5 + x^6$. This may be called the one-die series, and each term or (x) represents one face of the die or one of the six different, equally likely ways in which one die may fall. The plus signs are merely convenient bounds setting off the divisions between the different kinds of chances or opportunities and have no significance in the chance interpretation, although these signs do serve a mathematical purpose in the determination of the chances for a number of dice in combination. The coefficient of each term of this expression (understood to be unity) indicates the count of ways, while the exponent of each (x) represents the numeral or score on the face of the die. This symbolism means that there is one way to get a score of (1), one way to get a score of (2), one way to get a score of (3), etc., throughout the whole series of numbers carried by the various faces. Assuming that the numerals, which are accidentally exposed by the throw of two dice, are added to arrive at a score; this sort of coincidence may be symbolized by expanding the one-die series to the second power; that is, by multiplying the same by another such series symbolizing a second die. In this process, the exponent representing the numeral on each face of the first die will be added to the exponent representing the numeral on each face of the second die, and the coefficient of each term of the result will total the count of ways in which each new exponent or additive score may be obtained. This result may be called the two-dice series and it reads as follows:

$$x^2 + 2\,x^3 + 3\,x^4 + 4\,x^5 + 5\,x^6 + 6\,x^7 + 5\,x^8 + 4\,x^9 + 3\,x^{10} + 2\,x^{11} + x^{12}$$

This symbolism means that there is but one way to get a score of (2), two ways to get a score of (3), three ways to get (4), etc., throughout the whole series, ending in two ways to get a score of (11) and but one way to get a score of (12). The actual process of carrying out the multiplication of two single-die series to arrive at this result, seems to afford the most illuminating approach to an adequate conception of chance that has ever been devised.

The power to which the single-die series is raised, in this process, symbolizes the number of dice in combination, and this holds true for any number of dice falling in fortuitous coincidence. For four dice in haphazard addition, the chance is symbolized by expanding the single-die series to the fourth power, or the two dice series to the second power. In this expansion, the total of the ways to fall in combination, must always equal the number of faces on one die raised to the power of the number of dice in combination, or in the case of four dice, six to the fourth power of 1 296 ways. The resulting scores and counts of ways for four dice scored by addition are shown in Table 2.

It must be obvious that the dice may be scored by multiplication of the numerals accidentally exposed, without violating any of the dictates of logic or reason. In order to typify multiple scoring of the dice, it is necessary to violate the strict principles of algebra in the process of expanding to the power of the number of dice. In strictly proper expansion, whenever any two terms are multiplied together, the exponents are added, and this addition symbolizes the addition of these two dice numerals to arrive at a score. It must be apparent then, that the purpose of scores by multiplication may be attained by simply multiplying the exponents in each of these operations, instead of adding. The number of products or ways of combination, in this process, will be exactly the same as the number of sums or ways of combination with exponents added, but the multiplication of exponents or dice numerals gives rise to entirely different scores and radically different coefficients or counts of ways for each sort of exponent or dice score.

TABLE 2.—Four Dice in Haphazard Combination.

By Addition:		By Multiplication:	
Score.	Ways.	Ways.	Score.
4	1	530	1 to 62
5	4	285*	63 to 124
6	10	148	125 to 186
7	20	98	187 to 248
8	35	65	249 to 310
9	56	46	311 to 372
10	80	30	373 to 434
11	104	24	435 to 496
12	125	16	497 to 558
13	140	18	559 to 620
14	146*	5	621 to 682
15	140	12	683 to 744
16	125	4	745 to 806
17	104	4	807 to 868
18	80	6	869 to 930
19	56	....	931 to 991
20	35	....	992 to 1 052
21	20	4	1 053 to 1 113
22	10	....	1 114 to 1 174
23	4	....	1 175 to 1 235
24	1	1	1 236 to 1 296

* Median or middle of count of ways.

In this multiplication of exponents or dice numerals, the least score for four dice is the product of four ones or unity while the maximum score is the product of four sixes or 1 296. This range, from 1 to 1 296 for four dice in multiplication, is comparable to the range from 4 to 24 for four dice in addition, and in order to show a fair comparison of these two styles of count distribution, the multiple dice scores have been divided into twenty-one classes of magnitudes corresponding to the twenty-one additive dice scores. The count of ways for each of these classes or arrays of multiple scores for four dice have been totaled and given in Table 2 so as to show the relative scatter of these two different styles of haphazard combination. The total of all the different arrays of count of ways will be seen to be exactly the same for either additive or multiple combination (or 1 296), while the count distribution has very different characteristics for these two styles of combination.

A comparison of the distribution of ways by haphazard addition, as shown in Table 2, with the count distribution, N', of observations in Fig. 12 against the two axes of X and Y, suggests that the principles of the Gaussian law may be said to be fairly applicable to the problem illustrated. There are, however, possible limitations to the capacity of the Pearsonian system of correlation that it seems well to weigh. Considering the distribution of the count of ways resulting from the haphazard multiplication of four dice, as shown in Table 2, it becomes apparent that it would be futile to attempt to reconcile this style of scatter by any such methods.

TABLE 3.—FOUR DICE IN A COMBINATION OF QUANTITIES OF COMMON PROBABILITY KIND, OR ALWAYS INVOLVING A COINCIDENCE OF FOUR OF A KIND.

	BY ADDITION (REGENERATIVE).		BY MULTIPLICATION (GEOMETRIC).	
	Score.	Ways.	Ways.	Score.
4 ones.................	4	1	1	1
4 twos..................	8	1	1	16
4 threes................	12	1	1	81
4 fours..................	16	1	1	256
4 fives...................	20	1	1	625
4 sixes..................	24	1	1	1 296

The possibility of causes in multiplication is not the only prospective difficulty which may be encountered. A great number of the contributions to a given result in Nature, in turn, may originate in some common cause or circumstance, and the values combined are thus of the common probability kind instead of the haphazard probability kind; that is, they display a style of combination analogous to a dice score of 4 ones, 4 twos, 4 threes, etc. In the case of elements combining by addition, this results in a magnification of the trend of magnitudes for the original elements, whatever that may be; and in the case of elements combining by multiplication, the results must bear a geometric relation to the sources. These two phases of the possible ways of combination are illustrated in Table 3.

The importance of the distinctions drawn between these different styles of combination, has been previously suggested by the writer* in studies of rainfall and run-off expectancy. These studies are still too fragmentary but, inasmuch as the literature of the subject extending back over a period of more than a century seems to be unaccountably silent concerning these aspects, it seems wise to call attention at this time to the possible revolutionary effect on correlation conceptions that these distinctions may involve.

THEODORE HATCH,† ESQ. (by letter).—The engineer is frequently confronted with the problem of determining the relation between two variable measures. There are several methods of doing this, varying in their accuracy from the simple, direct method of fitting the curve by eye, to the more elaborate method of Least Squares. To obtain the equation of the curve, a straight-line relationship between the variables or functions of the variables is usually sought. Although a visual examination of the extent of scatter or deviation of the points from the straight line of best fit gives some idea of the closeness of fit, the degree of relationship is not fixed. This disadvantage is overcome when correlation is made the basis of analysis. This approach possesses several advantages not generally appreciated by engineers.

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* *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 909 to 919.

† Instructor in San. Eng., Harvard Univ., Cambridge, Mass.

Two measures are said to be correlated* when, a series of the first measure of definite sizes being selected, the means of the corresponding sizes of the second measure are found to be a function of the first. If they are independent of the first measure, the measures are said to be non-correlated. A fundamental requirement of this method of analysis is that the relationship be linear, or capable of being rectified to a linear function. Low correlation, therefore, does not necessarily mean absence of relationship; it may imply, simply, the absence of linear correlation. Thus, when $y = k\,x^2$, there may be low correlation between y and x, but high correlation between log y and log x, since in the latter form the function is linear. In order to meet this difficulty, a test for linearity has been developed.

By plotting the means of the second measure against the selected sizes of the first, and *vice versa,* two lines known as regression lines are obtained. When perfect correlation exists, these two lines are straight and they coincide. When the two measures are mathematically independent, the regression lines assume positions perpendicular to each other. As the degree of mathematical relationship varies from complete independence to perfect correlation, the angle between the regression lines decreases from 90 to 0 degrees. The coefficient of correlation, defined by the author, finds its basis in this fact. It can be shown that the slope of the regression line of Y on X has the value:

$$b_y = r\,\frac{\sigma_y}{\sigma_x} \qquad \text{(9)}$$

and of X on Y,

$$b_x = r\,\frac{\sigma_x}{\sigma_y} \qquad \text{(10)}$$

when $r = 1.0$, b_y becomes the reciprocal of b_x, and the two lines fall together. When $r = 0$, $b_y = 0$ and $b_x = 0$, and the regression lines are perpendicular to each other.

The coefficient of correlation may have any value between + 1.0 and — 1.0, a positive value indicating a direct relation between the two measures and a negative value an inverse relation. It is easily shown that r can never have a value greater than + 1.0 nor less than — 1.0. The following arbitrary scale of relationship in terms of the coefficient has been suggested:

Perfect correlation..................	$r =$	± 1.0
High correlation....................	$r =$	0.75 to 1.0
Considerable correlation............	$r =$	0.50 to 0.75
Moderate correlation................	$r =$	0.25 to 0.50
Low correlation.....................	$r =$	0.25

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* See "Medical Biometry and Statistics," by Raymond Pearl, W. B. Saunders Co., Philadelphia, 1923.

An additional measure of the degree of relationship is given by the probable error of the coefficient of correlation. By definition, it is a measure of the reliability of the coefficient itself, this reliability increasing as the ratio of the coefficient to its probable error increases. In practice, the coefficient is not considered to be statistically significant unless it has a value greater than six times its probable error. Thus, a value of $r = 0.3976 \pm 0.0103$ indicates moderate, but nevertheless significant, correlation; whereas a value of $r = 0.7693 \pm 0.2610$ indicates high correlation, but has little statistical reliability, since the coefficient is only three times its probable error.

It should be understood that correlation does not necessarily imply casual relationship. The coefficient of correlation is simply a measure of the mathematical relation existing between two variables. If this fact is kept in mind, many erroneous conclusions may be avoided.

From a knowledge of the coefficient of correlation, it is possible to write the equations of the mean regression lines of Y on X and X on Y, and it can be shown that the curves and equations thus obtained are identical with those given by the Method of Least Squares.

The coefficient of correlation may also be used as a measure of the relative value of various "yardsticks" proposed as indices of a certain condition. An index that does not take into account all factors involved will show low correlation with the condition to be measured; conversely, an index made up of all factors will be highly correlated with that condition.

This point is well illustrated in a recent paper dealing with the comparative values of "cooling power" and "effective temperature" as measures of conditions of comfort in ventilation.* Taking pulse rate as a measure of body comfort, the following coefficients were obtained for the two methods:

Pulse rate and effective temperature...... $r = 0.7845 \pm 0.029$
Pulse rate and cooling power............. $r = 0.5632 \pm 0.052$
Difference .... $= 0.2213 \pm 0.0595$

These results indicate high correlation between pulse rate and effective temperature and lower correlation between cooling power and pulse rate. Moreover, the coefficient of correlation between effective temperature and pulse rate is twenty-seven times its probable error; whereas in the case of cooling power and pulse rate, the coefficient is only ten times its probable error. Hence, it may be concluded that effective temperature constitutes a better index of body comfort (as measured by pulse rate) than cooling power.

The preceding statements have to do with simple correlation between two related variables. Not infrequently, however, it is necessary to determine the laws of relationship between three or more variables. In laboratory investigations, this is done by controlling certain of the variable factors in each experiment. In dealing with natural phenomena, this procedure is not possible, but it can be done mathematically by methods of partial correlation. The coefficient of partial correlation gives the degree of relationship between two variables, when all others are held constant by the mathematical process, and can be expressed, as follows:

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* "Physiological Reactions of Resting Subjects to Cooling Power and Effective Temperature," by J. Argyll Campbell and T. C. Angus, *Journal of Industrial Hygiene*, December, 1928, 10, 331.

$$r_{12.34\,\ldots\ldots\,n} = \frac{r_{12.34\,\ldots\,(n-1)} - [r_{1\,n.34\,\ldots\,(n-1)}]\,[r_{2\,n.34\,\ldots\,(n-1)}]}{[1 - r^2_{1\,n.34\,\ldots\,(n-1)}]^{\frac{1}{2}}\,[1 - r^2_{2\,n.34\,\ldots\,(n-1)}]^{\frac{1}{2}}} \ldots (11)$$

The subscripts indicate that the correlation is between Measures 1 and 2, with Measures, 3, 4. . .n held constant. For three variables, the expression becomes:

$$r_{12.3} = \frac{r_{12} - r_{13} \cdot r_{23}}{(1 - r_{13}{}^2)^{\frac{1}{2}}\,(1 - r_{23}{}^2)^{\frac{1}{2}}} \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

The following example will illustrate the application of Equation (12). The effect of altitude and distance from the ocean upon the annual rainfall in San Diego County, California, are to be determined from the data given in the records of the various rainfall stations.[*] By simple correlation, the following coefficients are obtained:

Rainfall and altitude, $r = +0.8779 \pm 0.2346$
Altitude and distance, $r = +0.8747 \pm 0.354$
Rainfall and distance, $r = +0.6172 \pm 0.0934$

From these coefficients of simple correlation, it seems that (*a*) rainfall increases with altitude; (*b*) altitude increases with distance; and (*c*) rainfall increases with distance. The increase of rainfall with distance, however, is actually due, not to the effect of distance from the ocean, but to the fact that, with increasing distance, the altitude is greater. To take this into account, the coefficients of partial correlation may be used. These are found to be as follows:

Partial coefficient between rainfall and altitude, $r = +0.887$
Partial coefficient between rainfall and distance, $r = -0.206$

These coefficients show high positive correlation between rainfall and altitude when distance is held constant and rather low negative correlation between rainfall and distance when altitude is held constant. This inverse relationship between rainfall and distance which is obtained by partial correlation is in accordance with the laws of rainfall.

From a knowledge of the coefficients of partial correlation, it is possible to write the equation of the relationship between the variables. For three measures, the equation has the form:

$$X_1 = \left[\beta_{12.3}\,\frac{\sigma_1}{\sigma_2}\right] X_2 + \left[\beta_{13.2}\,\frac{\sigma_1}{\sigma_2}\right] X_3 + M_1 - a\,M_2 - b\,M_3 \ldots. (13)$$

in which, X_1, X_2, and X_3 are the measures; M_1, M_2, and M_3 are their means; σ_1, σ_2, and σ_3 are their standard deviations; and

$$\beta_{12.3} = \frac{r_{12} - r_{13} \cdot r_{23}}{(1 - r_{23}{}^2)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

$$\beta_{13.2} = \frac{r_{13} - r_{12} \cdot r_{23}}{(1 - r_{23}{}^2)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

[†] Data taken from Mead's "Hydrology," p. 294, McGraw-Hill Book Co., N. Y., 1919.

$$a = \beta_{12.3} \frac{\sigma_1}{\sigma_2} \quad \cdots\cdots\cdots\cdots \quad (16)$$

$$b = \beta_{13.2} \frac{\sigma_1}{\sigma_3} \quad \cdots\cdots\cdots\cdots \quad (17)$$

For this example, when X_1 is the annual rainfall, in inches, X_2 is the altitude, in feet, and X_3 is the distance from the ocean, in miles:

$$X_1 = 0.00796\, X_2 - 0.628\, X_3 + 15.1 \quad \cdots\cdots\cdots \quad (18)$$

These single examples are cited in order to illustrate the use of correlation in experimental work and investigation and, in particular, to suggest to engineers the possible advantages of applying this method to the analysis of their problems.

B. F. Jakobsen,[*] M. Am. Soc. C. E. (by letter).—The author is to be complimented for submitting his graphical method for determining the coefficient of correlation, *r*, and its probable error. The method presented is certain to appear at a disadvantage, due to the small scale made necessary in reproducing it in a paper. The small scale gives the impression of a complexity that probably would not occur if the scale was large, as it would be, in actual use. Fig. 12 gives a clear illustration of the entire problem, including the analytical or arithmetical solution for those who may prefer that. Mr. Arne Fisher has pointed out[†] that, when the probable error cannot be determined, no great importance can attach to the result, since the probable error may be as large, or even larger, than the result itself.

The writer believes engineering will benefit, if engineers will acquire the habit of analyzing their "facts" by appropriate mathematical methods, such as those outlined in this paper. The truth is that the writer leans somewhat to the opinion that "facts" may not become "facts" until they are so analyzed by proper methods, or, at least, that they are not useful facts in a scientific sense, until analyzed and properly qualified.[‡]

The main problem is to determine the probable error, for it gives information as to the reliability of the observations and the theory. If the probable error is larger than the actual errors likely to have been made in measuring, it signifies that the equation assumed does not represent the phenomenon under investigation with sufficient accuracy. For simple problems the method of least squares suffices, but for more complicated phenomena, correlation must be used.

When Kepler was studying the observations made by Tycho Brahe, in order to determine the orbits of the planets, he assumed at first a circular path. He found "errors" of 7′ and 8′, but his confidence in Tycho's measurements was so implicit that he refused to believe that the actual errors of observations were that great. He then tried an elliptic orbit, assuming the sun to be located in one focus, and found "errors" of only 1′; this led to the discovery of the now famous Kepler's laws of planetary motion, from which Newton derived his law of gravitation.

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[*] Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

[†] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 83.

[‡] See Karl Pearson, "Grammar of Science" (1911), and the various popular writings of the great French mathematician, Henri Poincaré, "Science and Hypothesis," "The Value of Science," "Science and Method," and "Dernières Pensées".

The writer has analyzed the results of tests on the Stevenson Creek Dam and has found for the arch at Elevation 30, that p, the water load carried by the arch, was 9.80, and its probable error, ± 0.50; for a factor, k, determined by the shrinkage, he found k equal to -681 ± 480.* The probable errors show that the value found for p is quite reliable, while the value for k is much less so.

Professor Karl Pearson has shown† that experience does not and cannot prove causation, which is an assumption supplied by Man and then projected into the world of phenomena; but experience does show correlation. The coefficient of correlation with its probable error makes it possible to determine the degree of correlation and to compare one experience with another. A great many important problems, now generally drowned in a mass of words, can be attacked intelligently and effectively only by means of the coefficient of correlation. Such problems as the effect of strikes on wages, the effect of concentration of industry in large establishments on wages and working conditions, etc., can be unraveled only by appropriate mathematical treatment.‡

Every engineer must often have been vexed or amused when reading the voluminous opinions offered by successful business men, politicians, or chiefs of police departments, on the causes of automobile accidents. This problem might be profitably studied by a few men properly trained and equipped, and such a study would hinge largely on the determination of the various coefficients of correlation.

Leonardo da Vinci wrote:§ "He who scorns the certainty of mathematics will not be able to silence sophistical theories which only end in a war of words." Although written more than four centuries ago, and in spite of considerable progress made in the meantime, this statement applies as if it were written only yesterday.

WESTON S. EVANS,‖ ESQ. (by letter).—The trend of the discussion seems to indicate that the writer considered the graphical solution to be superior to the arithmetical solution for solving correlation tables. This is not the case. Mr. Dunlap was correct in stating that "the advantage of the graphical solution is in the understanding and grasp derived of the problem of correlation while learning about this method of analysis."

If a numerical answer only is sought, plain arithmetic is the best method by which to obtain it. Means and standard deviations are very easily determined graphically, and, if it becomes necessary or desirable to produce a finished drawing to scale, the extra time required to complete the solution is not great. The four hours required to solve the table in Fig. 2 was the time necessary to draw Fig. 12. If the finished drawing is not required, the graphical solution is entirely out of place and 20 min. is sufficient. The writer has solved many similar tables in that length of time, after making the table which is a part of the graphical solution. If speed is the controlling factor, then punching, sorting, tabulating, and calculating machines are the implements which should be used. The average engineer, however, has many lessons to learn before he is able to use such devices in solving his statistical problems.

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* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1243.

† "Grammar of Science," by Karl Pearson, 1911.

‡ See, for example, "Law of Wages," by Prof. Henry Ludwell Moore, 1911.

§ "A Short History of Science," by W. T. Sedgwick and H. W. Tyler, Macmillan Co., 1927.

‖ Asst. Prof., Civ. Eng., Univ. of Maine, Orono, Me.

Mr. Dunlap states that it is better to assume the origin as near as possible to the center of gravity of the table than near one corner in order to avoid negative numbers. For pencil and paper solution this is probably correct. The writer has been accustomed to adding machines on which the negative sign becomes a source of trouble and the magnitude of the numbers is of little consequence. Furthermore, when the data are tabulated on cards for use in sorting and tabulating machines, the code for any variable must start with the lowest values, and hence the origin will be in the lower left-hand corner. Although the choice of origin is optional, the writer believes that the best practice is to avoid negative signs.

The explanation of the regression equations as given by Mr. Strachan is worthy of some elaboration. In the two regression formulas, $x = 0.90\,y + 11.59$, and $y = 0.90\,x + 9.74$ (Equations (5a) and (7a)), it becomes evident that, although both apply to the same data, a given value of y gives evident values of x. For example, if $y = 100$ in Equation (5a), x will equal 102.22; but it cannot be said that if $x = 102.22$, y will equal 100, since, from Equation (7a), y would equal 101.00. As r decreases, the difference between the two equations will increase. In Equation (5a), x is the mean of all 7-day ratios that will occur for any given 28-day ratio; while in Equation (7a) y is the mean of all 28-day ratios that will occur for a given 7-day ratio. This principle, so seldom applied to engineering data, is one of the best answers to the question, "Why a correlation table?"

The discussions by Messrs. Moyer and Hatch are somewhat aside from the problem of solving a correlation table, but are nevertheless very interesting. Mr. Hatch has presented very well the possibilities of partial and multiple correlation.

Mr. Jakobsen has well expressed the value of determining the probable error of observations, estimates, and mathematical conclusions. The writer fully agrees that such statistical methods as have been discussed, if applied to many problems relative to engineering work, would produce definite conclusions rather than long drawn-out discussions which frequently lead to nothing.

In closing, the writer wishes to emphasize that means, standard deviations, and correlation coefficients are constants which are necessary to establish facts in regard to the relationship existing between pairs of variables. For simple relationship between two variables the simple correlation coefficient is satisfactory; but as relationships become intricate and many, the partial and multiple coefficients, as outlined by Mr. Hatch, will relate facts that can be obtained in no other way. Whether the correlation table is solved graphically or arithmetically, its use in the engineering field should become more common and thereby give a better understanding of the relationship between cause and effect.

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Paper No. 1748

EXPERIMENTS TO DETERMINE RATE OF EVAPORATION FROM SATURATED SOILS AND RIVER-BED SANDS[*]

By Ralph L. Parshall,[†] Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. Samuel Fortier, A. L. Sonderegger, Harry F. Blaney, N. W. Cummings, Carl Rohwer, Ivan E. Houk, and Ralph L. Parshall.

Synopsis

Much study has already been given by various investigators to the loss of moisture from soils by evaporation, as well as the effect of mulches in retarding or preventing this loss.[‡] The moisture content of the soil, which is held in capillary form, is least at the surface where the evaporation occurs, and leaves there a deposit of such salts as were dissolved in the soil moisture. Heavy Western soils, which usually carry alkaline salts, have the greatest capillary attraction; that is, such soils will bring moisture to the surface from greater depths but at a slower rate than the sandy or the lighter soils.

This paper is based on a study of the evaporation loss from soils contained in tanks under like exposure, the materials ranging from coarse river sand to heavy dark alkaline soil, with the water-table maintained at 1, 6, and 12 in. beneath the surface. It is not the writer's intention to present conclusive data as to the loss by evaporation from moist soil surfaces, but rather to describe the methods and procedure followed in this study and to submit such information

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Note.—The Special Committee on Irrigation Hydraulics has selected the subject of "Evaporation from Soils" as one of ten for study and research. This paper was submitted to the Committee by its author, and the Committee recommended its publication (see Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 173).

[*] Published in April, 1929, *Proceedings*.

[†] Irrig. Engr., U. S. Dept. of Agriculture, Fort Collins, Colo.

[‡] Progress Report of the Special Committee on Irrigation Hydraulics, *Proceedings*, Am. Soc. C. E., March, 1928, Society Affairs, p. 177.

as is now available as an indication of the general trend of the relations existing between the various soils and conditions as well as supporting the general conclusions drawn.

Description of Apparatus

Heretofore, the evaporation loss from soils has been determined by weighing the soil mass contained in tanks at stated intervals, the difference in weights equalling the loss by evaporation for the period. The problem of maintaining a constant and fixed position of the water-table in the soil, as well as that of determining the rate of loss due to evaporation, was solved, in the experiments herein reported, by the use of the principle of the Mariotte flask. In this regulating apparatus (Fig. 1) a calibrated air-tight drum, *D*, serves as the supply reservoir and is connected to a buried soil tank by a ½-in. supply pipe. A glass tube is mounted beside the supply reservoir and is connected to the system at both top and bottom so that the water level in the drum may be observed. The quantity withdrawn is indicated by differences read on the meter stick.

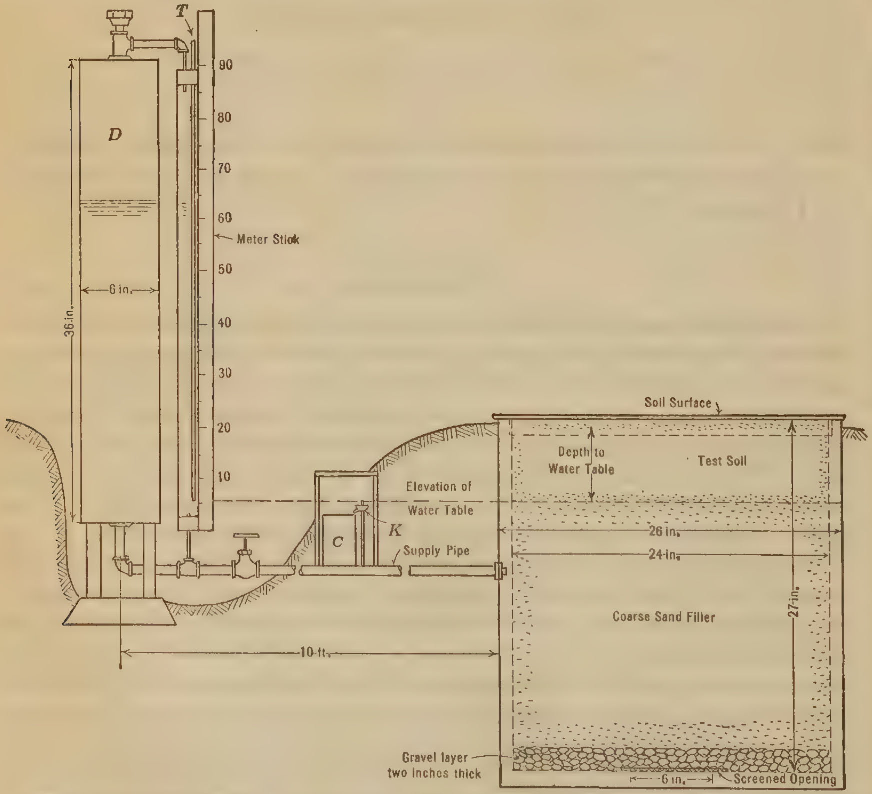
T
90
80
70
60
50
40
30
20
10
Meter Stick
D
6 in.
36 in.
C
K
Elevation of Water Table
Supply Pipe
10 ft.
Soil Surface
Depth to Water Table
Test Soil
26 in.
24 in.
27 in.
Coarse Sand Filler
Gravel layer two inches thick
6 in.
Screened Opening

Fig. 1.—Mariotte Regulation Feed to Supply Evaporation Loss from Soil Tanks.

Previous experience with the apparatus indicated that variations of temperature of the partial vacuum in the supply reservoir changed the vapor pressure to such an extent that dependable readings were not possible for short periods. In order to reduce the effects of outside temperature changes to a minimum all supply reservoirs are well covered with corrugated asbestos board next to the drum, *D,* and this is covered with one layer of building paper. Each individual set of apparatus is mounted in a wooden housing, and the space between the insulated drum and the sides of the housing is filled with clean dry wood shavings. A hinged lid at the top of the housing is provided for convenience in refilling the supply reservoir.

The valve in the supply pipe is necessary to prevent over-charge of the soil tank when the supply reservoir is being refilled. To insure against the water-table in the soil tank being overcharged, either accidentally or by excess water entering from the supply reservoir as an effect of temperature, a vertical tube, *K* (Fig. 1), is attached to the supply pipe. The top end agrees in elevation with the plane of the water-table in the soil tank. Excess water in this tank is wasted through this small tube into the container, *C,* where it is measured, and the indicated evaporation loss corrected accordingly.

The outside of the housing is whitewashed. A bulb of a thermograph has been sealed inside the supply reservoir of Tank No. 17 (Fig. 2) to indicate the variation of temperature. Covers are provided for all tanks to shield them against rain.

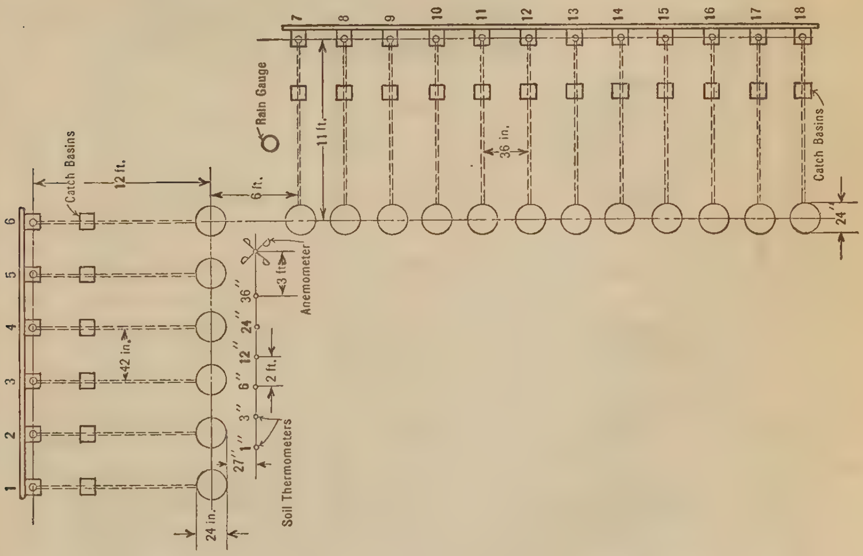

FIG. 2.—LAYOUT PLAN OF APPARATUS FOR MEASURING EVAPORATION FROM MOIST SOILS.

The soil tank consists of two shells, the outer constituting a water-tight container, while the inner, containing the soil, is supported at the top by an angle-iron rim so that a 1-in. space separates the inner tank from the outer at the sides and bottom. A hole, 6 in. in diameter, through the bottom of the inner tank, is covered by a coarse screen, and immediately above this is a 2-in. layer of gravel. Except in two tanks, coarse sand is placed on the gravel layer

TABLE 1.—AVERAGE EVAPORATION LOSS FROM SOILS IN TANKS, AND RATIO OF EVAPORATION FROM SOILS TO THAT FROM A FREE WATER SURFACE FOR THE PERIOD INDICATED.

Tank No.	Loss, September 3 to 8, 1925, in inches.	Ratio, percentage.	Loss, September 8 to 13, 1925, in inches.	Ratio, percentage.	Loss, September 24 to 30, 1925, in inches.	Ratio, percentage.	Mean ratio, percentage.	Depth of water-table below surface, in inches.	Remarks.
1	0.172	85	0.180	85	0.144	86	85	1	Before 1927, fine river sand mixed with silt, from Arkansas River, at Lamar, Colo.; after 1927, coarse clean river sand, from Poudre River, near Fort Collins, Colo.
2	0.162	80	0.169	80	0.136	81	80	1	Before 1927, fine sand mixed with dark-colored silt and mud, from Arkansas River, at Las Animas, Colo.; after 1927, mixing grade, clean river sand, from Poudre River, near Fort Collins, Colo.
3	0.171	85	0.182	86	0.156	93	88	1	Fine sand mixed with silt, from Arkansas River, at Avondale, Colo.
4	0.177	88	0.191	90	0.150	89	89	1	Fine sand mixed with silt, from Arkansas River, at Rocky Ford, Colo.
5	0.095	47	0.081	38	0.061	36	40	1	Heavy adobe alkali soil taken below grass roots in marsh, 3 miles south of Fort Collins, Colo.
6	0.166	82	0.173	82	0.137	82	82	1	Clay loam soil from experimental plots on the campus of Colorado Agricultural College, Fort Collins, Colo.
7–8	0.202	100	0.212	100	0.168	100	100	Free water	Evaporation from free water surface, mean value.
9	0.061	30	0.047	22	0.031	18	23	6	Heavy adobe alkali soil taken below grass roots in marsh, 3 miles south of Fort Collins, Colo.
10	0.186	92	0.197	93	0.149	89	91	6	Clay loam soil from experimental plots on the campus of Colorado Agricultural College, Fort Collins, Colo.
11	0.064	32	0.058	27	0.064	38	32	6	Fine sand mixed with silt, from Arkansas River, at Rocky Ford, Colo.
12	0.159	79	0.154	73	0.115	68	73	6	Coarse clean river sand, from Poudre River, at Fort Collins, Colo.
13	0.176	87	0.168	79	0.141	84	83	6	Medium grade of clean river sand, from Poudre River, at Fort Collins, Colo.
14	0.040	20	0.029	14	0.016	10	15	12	Heavy adobe alkali soil taken below grass roots in marsh, 3 miles south of Fort Collins, Colo.
15	0.183	91	0.181	85	0.148	88	88	12	Clay loam soil from experimental plots on the campus of Colorado Agricultural College, Fort Collins, Colo.
16	0.088	44	0.080	38	0.062	37	40	12	Fine sand mixed with silt, from Arkansas River, at Rocky Ford, Colo.
17	.....	....	.....	....	.....	....	....	12	Coarse clean river sand, from Poudre River, at Fort Collins, Colo.
18	0.142	70	0.140	66	0.114	68	68	12	Medium grade of clean river sand, from Poudre River, at Fort Collins, Colo.

up to the plane of the water-table, and on this porous material the test soil is laid. The water contained between the inner and outer shells of the tank is fed upward through the large hole in the bottom of the inner tank and thence through the soil to supply the moisture lost by evaporation at the surface. Loss of water by evaporation from the water surface between the two shells is prevented by sealing the top of the outer tank with heavy canvas and coal-tar. Tanks Nos. 1 to 6, inclusive, contain different types of soil, with the water-table at 1 in. below the soil surface. Tanks Nos. 7 and 8 contain water only. In Tanks Nos. 9 to 13, inclusive, the water-table is 6 in. beneath the surface, while in Tanks Nos. 14 to 18, inclusive, it is 12 in. beneath the surface. Six soil thermometers are placed parallel to the axis of Tanks Nos. 1 to 6 at depths ranging from 1 to 36 in. A Robinson anemometer and a standard rain-gauge are mounted near Tank No. 7. Tanks Nos. 7 and 8 are equipped with hook-gauges, reading to 0.01 in. A maximum thermometer is supported in the former and a minimum thermometer in the latter.

The Arkansas River sand in Tanks Nos. 1 and 2 was removed at the beginning of the 1927 season and coarse and medium river sand substituted. The materials in all other tanks used in 1927 were the same as those used in 1926 (see Table 1). A mechanical analysis of the materials in six of the tanks is shown in Table 2.

TABLE 2.—MECHANICAL ANALYSIS OF SOILS IN TERMS OF PERCENTAGE OF THE COMPLETE SAMPLE.

Tank No.	Fine gravel (2 to 1 mm.). (1)	Coarse sand (1 to 0·5 mm.). (2)	Sand (0·5 to 0·25 mm.). (3)	Fine sand (0·25 to 0·10 mm.). (4)	Very fine sand (0·10 to 0·05 mm.). (5)	Silt (0·05 to 0·005 mm.). (6)	Clay (0·005 to 0·0001 mm.). (7)
3	0·6	1·1	2·0	27·8	62·2	5·7	0·6
4	5·4	3·2	2·3	21·8	61·5	4·9	0·9
5	0·4	4·4	7·2	31·5	33·6	10·8	12·1
15	0·8	2·1	2·9	20·0	42·8	25·5	5·9
17	35·6	37·0	16·8	7·1	1·5	1·4	0·6
18	7·5	20·0	29·5	36·0	5·0	1·0	1·0

The alkaline soil from Tank No. 5 and the loam soil from Tank No. 6, which is identical to that contained in Tank No. 15, were analyzed to determine the percentage of water-soluble materials. This was found to be 0.261% and 0.153%, respectively. As usual, the analysis of these water-soluble portions present some difficulties; however, the salts present were the ordinary ones, such as sulfate of lime, magnesia, and soda. The chlorine content was low, a feature common to the alkalies in Northern Colorado. While the quantity of carbonate in Tank No. 5 was about the average, that in Tank No. 6 appeared to be greater. It seems to be a question as to how much manganese, phosphoric acid, alumina, and ferric oxide may be expected in the water-soluble portion.

All tanks are not filled in the same manner. In Tanks Nos. 1 to 6, the water-table is carried at a nominal depth of 1 in. Tanks Nos. 1 to 4, inclusive, are filled with coarse river sand to within about 4 in. of the top. The sample material, 2½ in. deep, is then added, which leaves about 1½ in. between the top of the soil and the rim of the tank. Tank No. 5, which contains the heavy alkali soil, was filled from the gravel layer in the bottom to a point within about 1½ in. of the rim. Tank No. 6 was also completely filled in a manner similar to Tank No. 5, with soil taken from the excavation made in installing the equipment. Tanks Nos. 9 to 13 were filled with coarse sand to 6 in. below the top or rim, above which was placed the test soil. Tanks Nos. 14 to 18 were likewise filled to a level 12 in. below the rim and topped with the test soil.

In the case of Tanks Nos. 5 and 6, which were completely filled with heavy soil, the intention was to ascertain what relative effect would be apparent when the water that was lost in evaporation was drawn through a greater depth of soil than that represented in the other tanks.

Tanks Nos. 1 to 6 were the first set installed. The angle-iron rim at the top of the inner tank was placed so that the inner tank was exposed above the ground surface about 1½ in. This arrangement was improved in Tanks Nos. 7 to 18, by banking soil around the tank as shown in Fig. 1.

The surface of the soil in all tanks was kept free of plant growth and in an undisturbed condition. The high water-table in Tanks Nos. 1 to 6 developed a slight surface scum of algæ or bacterial formation. This yellowish-brown coating is thin and more or less elastic. What effect it may have on the rate of evaporation is not known, but it is believed to be small. No doubt, copper sulfate added in small quantities to the surface of the soil would retard or prevent the growth, but because a natural condition was sought, the surface was left untouched.

The appearance and texture of the soil surfaces were observed on September 17, 1926. Tanks Nos. 1 to 6 were all yellowish-brown with a scum of growth about $\frac{1}{32}$ in. in thickness. This covering or membrane was somewhat elastic, but did not cover the entire surface; that is, it was in patches of various areas. The edges of these patches were in some cases rolled back. The layer of scum on the soil surface of Tank No. 6 was thin. A deposit of alkali showed on the surface of Tank No. 5. The soil surface in Tank No. 9 was a light gray color, showing some alkali. Small cracks appeared here and there, with the soil becoming dry and mealy.

The soil in Tank No. 10 was dark brown, with damp surface and patches of scum. A very thin scum showed on the soil surface of Tank No. 11, the soil being damp, with concentrated isolated alkali crystals, compact and dark colored. No alkali showed on Tanks Nos. 12 and 13. The surface was damp with a thin scum, generally greenish-brown in color.

The surface of Tank No. 14 was almost white with an alkali deposit, very irregular or uneven, dry and mealy, with a few small surface cracks. This surface, when touched, would crumble to a fluffy mass. No alkali appeared on the soil surface of Tank No. 15 which was damp, with a very slight, brownish scum, and of mealy texture. Concentrated alkali crystals were spread over one-half the surface area of Tank No. 16 which was damp, with slight scum; the soil was more compact. The surface of Tank No. 17 was clean, smooth, and dry. Tank No. 18 showed a damp surface, inclined to be slightly incrusted.

EFFECT OF DEPTH TO WATER-TABLE ON RATE OF EVAPORATION

Soil Tanks Nos. 1 to 6 were set up and filled in May, 1925, and Tanks Nos. 7 to 18 were installed in July, 1925. Because the filling of soil was uncompacted and the apparatus untried, only preliminary observations were made during this first season to ascertain the characteristics of the equipment. Fragmentary results of the comparison of the evaporation loss from the several moist soils and the free water surface are given in Table 1. This record covers only a part of September, 1925. The insulation for the Mariotte tanks was not sufficient at first to give reliable readings because of the variation of pressure of the water vapor in the space, *D* (Fig. 1). Considerable trouble was also caused by leakage of air into the Mariotte system. The instability of this equipment and the fact that the tanks were newly filled do not justify the placing of much dependence on these first results, although they show, in a general way, the trend of the evaporation loss as determined for the season of 1926. The period from September 14 to 23, 1925, was excluded because of the interference of rainy weather.

At the beginning of the season of 1926, the Mariotte tanks were completely overhauled and re-set. Thermograph records of the temperature of the partial vacuum within the supply reservoir (Tank No. 17) show a maximum daily range of 7° Fahr. and a minimum range of 1° Fahr. This range of temperature change is sufficient to affect materially the night and morning readings of the water level in the Mariotte supply reservoir. Theoretically, there can be no return of water to the supply reservoir from the soil tank as a result of reduction of the pressure of the water vapor in the space, *D* (Fig. 1). In order to investigate the apparent inconsistency of the action of the Mariotte regulation, special apparatus was set up by which the pressure of the water vapor in the space, *D*, could be greatly increased and decreased by changing the temperature. It was demonstrated that when a very slight decrease in temperature occurred, with no evaporation from the exposed sustaining or balancing water surface, an accumulation of water in the supply reservoir was possible. During a period of falling temperature, the pressure of the water vapor decreases slowly, and for a condition of balanced pressures a small bubble of air is held at the lower end of the small brass vent tube, *T* (Fig. 1). The surface tension holding this bubble in place operates as a seal at the lower end of the vent tube. Manifestly, if this tube is sealed, a decrease of pressure in the space, *D*, will cause water to flow from the soil tank into the reservoir of the Mariotte apparatus.

At first, it was thought possible to differentiate between the night and day losses of evaporation from the several types of soil, but when the data were studied thoroughly, no correlation of these losses appeared. The observations during the season of 1926 were taken at approximately 8:00 A. M., an hour when the air temperature was reasonably uniform for the weekly periods. For the season of 1927, the observations were taken at about 11:00 P. M.

Table 3 gives the average evaporation loss, in inches per day, for 1926 and 1927 and a comparison of losses from the soils with that from a water surface. During 1926, a complete evaporation record from a free-water surface was also observed in an area distant about 500 ft. northwest of the experimental apparatus. This is designated Tank *A* in Table 3.

Covers were provided to protect the tanks against rain, but nearly each week more or less rain fell on the exposed soil surfaces. It was found practically impossible to guard against this condition because all soil surfaces were being exposed to natural conditions. Experience with these tanks shows that a marked reduction in the evaporation loss occurs when the covers are in place.

The cooling effect of the rain on the soil increases the surface tension of the capillary moisture drawn up from the water-table. Rain water falling on the soil also dilutes the soil solution and if the solution is alkaline it increases the rate of evaporation. It is evident that, although adding moisture to the soil at the time, light showers may later cause a more rapid depletion of the moisture already within the soil. Messrs. F. H. King and J. B. Stewart,[*] have also noted this phenomenon.

TABLE 3.—AVERAGE EVAPORATION LOSS, IN INCHES PER DAY, FROM MOIST SOILS AND MEAN RATIO OF EVAPORATION FROM SOILS TO THAT FROM A FREE WATER SURFACE TAKEN AS 100.

Tank No.	Depth of water-table below surface, in inches.	JUNE 23 TO OCTOBER 20, 1926.		MAY 1 TO OCTOBER 30, 1927.		Depth of test soil, in inches.
		Evaporation loss, in inches.	Ratio.	Evaporation loss, in inches.	Ratio.	
1	1	0.169	99	0.166	106	3
2	1	0.166	96	0.171	109	3
3	1	0.180	100	0.159	101	3
4	1	0.178	100	0.165	105	3
5	1	0.132	75	0.131	83	Full depth of tank
6	1	0.143	83	0.136	87	Full depth of tank
7 and 8	Free water	0.173[*]	100	0.157	100	....
9	6	0.048	28	0.105	67	6
10	6	0.176	102	0.154	98	6
11	6	0.126	73	0.147	94	6
12	6	0.171	99	0.104	66	6
13	6	0.184	104	0.158	101	6
14	12	0.019	10	0.051	32	12
15	12	0.097	56	0.129	82	12
16	12	0.110	64	0.157	100	12
17	12	0.007	4	0.031	20	12
18	12	0.090	52	0.082	52	12
A[†]	Free water	0.193	111	.....	....	....

[*] This is based on fifteen weekly averages. The averages from some of the soil tanks are for shorter periods, hence are not directly comparable with that for Tank No. 8.

[†] Tank, 3 by 3 by 3 ft., located 500 ft. northwest of evaporation plots.

To study more in detail the effect on evaporation, of sprinkling the soil surface in quantities that approximate a shower of rain, two soil tanks, Nos. 17 and 18, were so treated. Fig. 3 shows the relation of loss from a free water surface and the losses from Tanks Nos. 17 and 18 during the sprinkled and unsprinkled periods. The quantities of water applied during the sprinkled period are shown in Table 4. For the unsprinkled period, the ratio of loss from the tanks when compared with that from the free water was 11% for Tank No. 17 and 32% for Tank No. 18. The corresponding ratios for the

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[*] "The Principles of Soil Management," by Messrs. T. L. Lyon and E. O. Fippin, p. 198, 1911.

TABLE 4.—RECORD OF WATER CONSUMPTION FOLLOWING SPRINKLING ON SOILS.

PERIOD.		TANK NO. 17.					TANK NO. 18.				
			Application.					Application.			
Number.	Week.	Loss per day for the week (scale reading),* in inches.	Date.	Total quantity, in inches.	Equivalent per 24 hours, in inches.	Total water consumed per day for the week, in inches.	Loss per day for the week (scale reading), in inches.	Date.	Total quantity, in inches.	Equivalent per 24 hours, in inches.	Total water consumed per day for the week, in inches.
12	July 17–24	0.019	July 21, 23	0.203	0.029	0.048	0.076	July 21, 23	0.203	0.029	0.105
13	July 24–31	+0.001	July 26	0.010	0.014	0.013	0.068	July 26	0.101	0.014	0.082
14	July 31–Aug. 7	0.004	None	None	None	0.004	0.066	None	None	None	0.066
15	Aug. 7–14	0.001	Aug. 10, 11, 12, 13, 14	0.506	0.072	0.073	0.046	Aug. 10, 11, 12, 13, 14	0.506	0.072	0.118
16	Aug. 14–21	0.011	Aug. 15, 18, 19, 20	0.405	0.058	0.069	0.088	Aug. 15, 18, 19, 20	0.405	0.058	0.146
17	Aug. 21–28	+0.004	Aug. 26, 27, 28	0.304	0.043	0.039	0.060	Aug. 26, 27	0.203	0.029	0.089
18	Aug. 28–Sept. 4	0.000	Aug. 30, Sept. 1, 3	0.304	0.043	0.043	0.118	Aug. 30, Sept. 1, 3	0.304	0.043	0.161
19	Sept. 4–11	+0.001	Sept. 6, 7, 8, 11	0.506	0.072	0.071	0.099	Sept. 7, 9	0.202	0.029	0.128
20	Sept. 11–18	+0.003	Sept. 12, 13, 14, 16, 17	0.506	0.072	0.069	0.120	Sept. 13, 17	0.202	0.029	0.149
21	Sept. 18–25	+0.001	Sept. 19, 20, 22, 24	0.405	0.058	0.057	0.092	Sept. 24	0.101	0.014	0.106
22	Sept. 25–Oct. 2	0.014	Sept. 29	0.101	0.014	0.028	0.061	None	None	None	0.061

* Plus sign indicates gain.

sprinkled period were 33% and 76%, respectively. To what extent the sprinkled water was held in the voids of the sands is not known, but because of the similarity of the rate-of-loss curves representing the soils to that representing the free water, the writer considers it fair to assume that the rate of evaporation loss for the sprinkled period was materially increased. The data given in Table 5 for Tank No. 17 indicate that the percentage of moisture held within the soil mass above the water-table was at or near the maximum capillary capacity, because the quantity used from the storage reservoir of the Mariotte regulation was very small, or showed an increase or an accumulation from the soil tank. Tank No. 18 drew liberally from the supply reservoir to make up for the increased loss due to evaporation.

Other experimenters studying evaporation losses from various types of soils have found results varying between rather wide limits. The problem involves too many factors to make possible any correlation which would show other than very general relations.

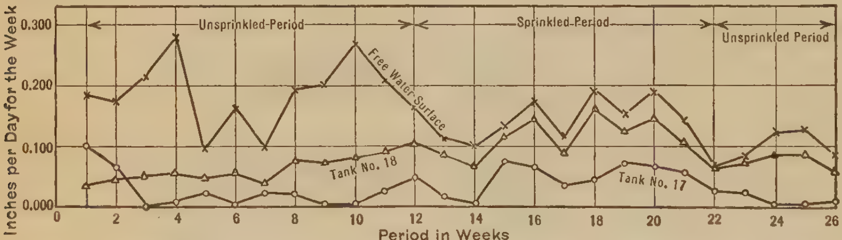

FIG. 3.—RELATION OF LOSS FROM FREE WATER SURFACE TO THE SPRINKLED AND UNSPRINKLED LOSSES FROM TANKS NOS. 17 AND 18.

In studying the evaporation losses from Yolo fine sandy loam, in tanks having diameters of 2 ft. 1 in., with the water-table at the surface and also at 1 in. below the soil surface, Samuel Fortier, M. Am. Soc. C. E., and Mr. S. Beckett, at Davis, Calif., found the following ratio of losses over a period of nine weeks, from August 2 to October 3:*

	Percentage.
Free water surface	100
Saturated surface..........................	84
Water-table at 1 in..........................	82

The late R. B. Sleight, Assoc. M. Am. Soc. C. E., working at Denver, Colo., with a river-bed sand, in tanks having a nominal diameter of 2 ft., found the evaporation losses from August 4 to October 16 to vary with depth of water-table, as follows:*

	Percentage.
Free water surface	100
Water-table at sand surface................	77
Water-table at 3 in..........................	69
Water-table at 6 in..........................	64.5
Water-table at 10½ in......................	57.7
Water-table at 24 in........................	11.3

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* These data were taken from "Use of Water in Irrigation," by Samuel Fortier, M. Am. Soc. C. E., New York, 1926.

Mr. Sleight also found, for similar conditions and for a fine sandy loam soil, the following variation of evaporation losses according to depth to water-tables:*

	Percentage.
Free water surface	100
Water-table at 4 in.	88.2
Water-table at 16 in.	79.8
Water-table at 28 in.	62.4
Water-table at 38 in.	33.0
Water-table at 43 in.	7.63
Water-table at 50½ in.	7.24

M. R. Lewis, M. Am. Soc. C. E., working at Twin Falls, Idaho, with a medium clay loam, in tanks having a diameter of 3 ft., found the following relation from May 27 to October 14:*

	Percentage.
Free water surface	100
Saturated surface	134

The writer has studied the evaporation from different types of soils at Rocky Ford, Colo., where the water-table was held at about 4 in. in soil tanks having a diameter of 2 ft. The findings for the period of August to November, 1924, inclusive, are arranged in Table 5.

TABLE 5.—EVAPORATION FROM DIFFERENT TYPES OF SOILS AT ROCKY FORD, COLORADO.

Location.	Material.	Depth of water-table below surface, in inches.	Ratio, percentage.
	Free water surface	....	100
Patterson Hollow	Heavy adobe alkaline soil	4.3	19
Experimental plots	Fine sandy loam	5.6	30
Arkansas River	Fine sand mixed with silt, Rocky Ford	4.1	80
Arkansas River	Fine sand mixed with silt, Avondale, Colo	3.8	77
Arkansas River	Fine sand mixed with clay and silt, Las Animas, Colo	4.1	58
Arkansas River	Fine sand mixed with silt, Lamar, Colo	3.8	68

The river sands in this group were substantially of the same general type. The sands in the Rocky Ford experiment were used as test soils in Tanks Nos. 1 to 4, as previously described (Table 1). The results obtained from June to October, 1926, for these same materials are as listed in Table 6.

Fig. (4) (*a*) and Fig. 4 (*b*) show graphically the trend of the evaporation loss from different soils with increasing depth of water-table. These losses are expressed as a ratio of that from a free water surface. In Fig. 4 (*a*) the four river samples of fine sand with the water-table at 1 in., show a loss equal to that from a free water surface. The loam and heavy adobe soils show a loss of 83% and 75%, respectively, for a depth of 1 in. to the water-table, as compared with that from free water. When the water-table is 6 in. below the soil surface, the Rocky Ford sand and the adobe soil show a reduction of loss, while the loam soil and both the medium and coarse river sands show a loss equalling that from the free water surface. These three soils throughout the period, June to October, showed a consistently high rate of evaporation. The reason for this persistent high rate of loss is not apparent.

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* These data were taken from "Use of Water in Irrigation", by Samuel Fortier, M. Am. Soc. C. E., New York, 1926.

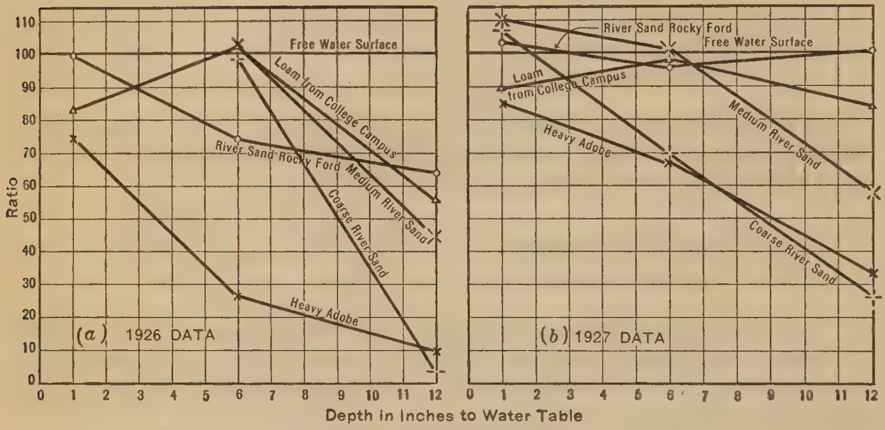

FIG. 4.—TREND OF EVAPORATION LOSS FROM DIFFERENT SOILS WITH INCREASING DEPTH OF WATER-TABLE.

CONCLUSIONS

It is believed that the method of using the Mariotte principle in obtaining the evaporation loss from moist soils, even though there are some apparent inconsistencies in its operation, gives more dependable results than the method of weighing. The results in Table 6 show very marked consistency for fine river sands, which are essentially identical, with the exception of the silt content. Those cited are as evidence of the dependability of the apparatus. The feature of automatic control is highly desirable.

TABLE 6.—EVAPORATION FROM DIFFERENT TYPES OF RIVER SANDS.

Material.	Depth of water-table below surface, in inches.	Ratio, percentage.
Free water surface	1	100
Rocky Ford, Colo., fine river sand	1	100
Avondale, Colo., fine river sand	1	100
Las Animas, Colo., fine river sand	1	96
Lamar, Colo., fine river sand	1	102

DISCUSSION

SAMUEL FORTIER,* M. AM. SOC. C. E. (by letter).—The United States is mainly indebted to two Federal agencies for its knowledge of water supplies. The U. S. Weather Bureau measures and records the precipitation as it falls from the clouds in the form of rain and snow, and the U. S. Geological Survey measures and records the quantity of water that flows to the sea. The incoming waters derived from precipitation, particularly throughout the western half of the United States, greatly exceed the outgoing waters conveyed in stream channels. There are large areas in the West in which no water derived from precipitation ever reaches the ocean in the form of run-off. This is true of the Great Basin comprising an area, exclusive of the Imperial Valley, of 138 000 000 acres. A near approach to this condition is also found in the basin of the Colorado River containing 155 000 000 acres, in which the normal run-off is less than 0.8 in. over the surface drained.

These references and others which might be given, show that in that vast region comprised in the seventeen Western States, only a small part of the total precipitation is carried off by streams, the greater part being converted into vapor which is soon wafted to distant localities by the prevailing winds. No Federal agency has been specifically charged with the collection of data having for its objective the determination of the quantities of water transpired from native vegetation and evaporated from uncultivated soil surfaces, for the apparent reason that until recently those who live in the humid part of the country have not been generally interested in the subject. On the other hand, to those whose occupation and abodes are in the arid and semi-arid regions—and they comprise one-fifth of the total—the subject is of vital concern. While they do not claim to be able to control, to any great extent, transpiration from plants or evaporation from soils, yet they do require a general knowledge of this enormous loss of water in order to plan aright for the utilization to the fullest extent possible of what remains of the water supplies.

Until recently the main water problems of the Nation have been confined to the territory west of the Missouri River because water was scarce, and because much more was needed for agricultural purposes. Westerners discuss water, investigate it, litigate over it, and end by criticizing the water decisions of the Courts. Now, however, many Western water controversies are having their counterparts in the Eastern States. The rapid increase in the use of water for hydro-electric plants and municipalities has enhanced its value and has led corporations and municipal and State Governments to transfer supplies from one drainage basin to another and from one State to another. This is quite enough to start trouble, and no one is wise enough to predict when or how it will end. In view of this situation, which is certain to become worse, and in the interests of public welfare generally, increased Federal and State funds might profitably be expended in acquiring more complete and reliable data concerning water supplies. Western experience has demonstrated that it is much cheaper and preferable in many ways to employ engineers to investigate water problems than lawyers to litigate over them.

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A field that is practically unexplored and one that should be investigated at an early date is to be found in the wide gap between the accumulated knowledge available concerning precipitation on the one hand and run-off on the other. To call attention to but two vacant spaces in this unexplored field of knowledge, there still remain in this country about 470 000 000 acres of virgin forests and cut-over and burned-over forested areas. Does this vast area transpire annually, on an average, 1 acre-ft., 2 acre-ft., or some other quantity of water per acre? Nobody knows and nobody has ever tried to find out. Furthermore, there are about 658 000 000 acres of unimproved pasture and range land. What is the loss of water by transpiration and evaporation from this area? Nobody knows.

In many respects, transpiration of water from plants differs from evaporation of water from soils, each having distinct and separate functions to perform, but in studying water supplies and their utilization it is seldom practical to consider them apart. Four drops of water are lodged in soil side by side. Three drops are absorbed by the rootlet of a plant, drawn up through its stem, and transpired by its foliage. The other drop is evaporated directly from the soil. All four reach the clouds, but by different routes. If one part of the land surface of the United States were bare soil and the other covered with vegetation, the rate and extent of evaporation from the bare part might be determined, but the almost inseparable twin problems of transpiration and evaporation would be found in the other. As a matter of fact there is very little soil in which plants of one kind or another do not grow at least a part of each year, and the quantity of water which is transpired by plants from a unit of such surfaces cannot well be segregated from that evaporated from bare soil in their immediate vicinity.

Since 1898 the Congress of the United States has appropriated relatively small sums of money to the Division of Agricultural Engineering for irrigation investigations. By co-operation with Western agricultural experiment stations and other State agencies, it has been possible, through the increased funds thereby made available, to carry on such studies in all the Western States. Several thousand experiments have been conducted in widely separated localities to determine the water requirements of all the most common field crops and the quantities of supplemental irrigation water needed for their proper growth. As a result of these investigations it is now possible to define with reasonable accuracy the irrigation requirement of a farm in any one of the Western States, provided knowledge is available of its location, climate, extent of irrigable area, character of soil, and the crops to be grown. Since, however, the irrigated land in the seventeen Western States constitutes less than 2% of the total, a knowledge of the quantity of water transpired and evaporated from irrigated farms does not help much in the solution of what takes place in respect to these natural functions on the remaining 98%, particularly as little or no data are available on which to base an estimate. This is true of native grass lands and lands covered with brush, shrubs, and virgin forests. It is likewise true to nearly the same extent of arable lands which are dry farmed. An erroneous impression prevails in attributing to soil evaporation the greater part of the vaporized waters escaping from uncultivated lands on which native plants, including forests, grow, whereas actually it is the native vegetation that transpires by far the greater part.

In passing from a consideration of the water yield of a Western drainage basin to the water problems of the irrigator, there are many ways in which evaporation from soils affects the irrigator's profits. In growing cultivated crops, such as orchard trees, cotton, sugar beets, or potatoes, the greater part of the ground surface is not only bare but is periodically watered and a large percentage of the water applied escapes as vapor into the atmosphere. To cite a single case, the average annual cost of irrigation water applied to the lands of 111 mutual water companies in California is $15.50 per acre. The yearly cost of applying this water is about $2.50 per acre, or $18.00 per acre in all. If 30% of this water is evaporated from bare soil, it represents a considerable loss to the grower. In an effort to find ways and means to lessen this and similar losses, the writer, aided by his associates and collaborators, sought to determine some of the underlying principles of soil evaporation. Dr. John A. Widtsoe, when President of the Agricultural College of Utah, and W. W. McLaughlin, now Associate Chief of the Federal Division of Agricultural Engineering, were the first to ascertain quantitatively by controlled experiments in tanks that the more moisture the soil contained the greater was the evaporation.

A series of experiments conducted by the writer confirmed this fact and led to the use of deeper furrows in applying water to cultivated crops. In the case of the orchard groves of California, the top layer of soil from 6 to 9 in. in depth, although it is the most fertile and best aerated part of the soil column of the root zone, serves mainly as a blanket to protect the moist soil beneath. Orchard roots which are lured into this zone by winter rains perish from drought during the long rainless summer. The writer also determined by experiments that the factors of temperature, wind movement, and humidity which influence the rate of evaporation from water surfaces were applicable to soil surfaces. It is well, however, to keep in mind that the governing factor in evaporation from soils consists in the percentage of moisture in the top layer. Under like conditions saturated soils may be said to evaporate about 90% of that from a water surface, but with the soil moisture at the wilting point of plants the evaporation may be reduced to little or nothing.

Another phase of evaporation which concerns rural communities of the Western States, is to be found in the condition of Western streams and their flood-plains during low-water periods. Some Western stream channels carry water so seldom that their natural function is not always apparent to the casual observer. Other streams, like the Platte, Rio Grande, and Arkansas, in the late summer months carry a small volume of water spread out in a thin sheet, while the greater part of the flood-plain is either dry or partly moistened. It occurred to the writer that if the loss of evaporated water from the shallow water surface as well as from the bare river bed material could be determined with a fair degree of accuracy, a basis could be laid to determine the economic feasibility of conveying the flow of the late summer months, either in an artificial canal or within a relatively small part of the flood-plain channel.

With this objective in mind, advantage was taken of a rather far-flung organization to collect twenty-seven typical 40-lb. samples of river-bed materials from each of sixteen stream beds. A mechanical analysis was made of each sample, and the percentage of pore space was also determined. The equipment was inadequate to make a separate evaporation test of each sample and, for this reason, the samples were combined into four groups ranging from the coarsest to the finest material. This grouped material was placed in double water-jacketed tanks of which the inner, with a few exceptions, was 23.5 in. in diameter and 48 in. deep. In an effort to duplicate Western stream-bed conditions in low-water periods, the water-table in each series of tank experiments was maintained at distances from the surface of 3 in., 12 in., and 24 in., and occasionally at other distances. Where the water-table was 3 in. below the surface, the results were fairly uniform for the different materials, varying from 66 to 77% compared with 100% from a water surface; but at depths of 12 and 24 in., the variations were greater. The experiments were carried on at the Denver, Colo., Field Laboratory and were in direct charge of the writer's assistant, the late R. B. Sleight, Assoc. M. Am. Soc. C. E.*

In this brief discussion of Mr. Parshall's paper, the writer has endeavored to show the relationship between evaporation from soils and transpiration from plants, and to give a few reasons for the acquisition of more knowledge of both branches of this twin problem.

A. L. SONDEREGGER,† M. AM. SOC. C. E. (by letter).—This paper describes a method of determining evaporation which apparently is vastly superior to the cumbersome method of finding losses by weighing the soil mass in tanks, and also to the method of maintaining a fixed water-table by daily closing. The experiments themselves present a valuable addition to the study of evaporation losses from moist areas.

In many regions of the semi-arid West, the elimination of evaporation from moist areas will form the last link in the chain of efforts to salvage the losses that pertain to the natural disposal of water supply. The factors which affect the problem are the total volume of evaporation losses available for recovery and the depth to which the water-tables must be lowered, with different soils, in order to stop capillary action.

As far as the experiments described by the author are concerned, it is apparent that the adjustment of the Mariotte flask to the special conditions presented by each test tank offers the most difficult problem with which the experimenter has to contend.

The results graphically shown on Fig. 4 permit of practical application. The sharp descent of the curve for the two extremes in soils—adobe and coarse sand—would indicate that, with a water-table 2 ft. below the surface, evaporation losses are eliminated. In clay soils, capillary action is effective, but very slow, while in the coarse sand, capillary tubes are short and interrupted. On the other hand, the high rate maintained by Rocky Ford sand, which occupies a position about half way between the two extremes, leads to the conclusion that the mixture of materials was such as to produce what might be called "optimum" capillary action, both as to time and rate. It must be remembered, however, that these soils were disturbed, having been removed from their original location.

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* *Journal of Agricultural Research*, Vol. 10, No. 5, 1917.

† Cons. Engr., Los Angeles, Calif.

In order to complete the evaporation curve, deeper tanks are required. Experiments to determine evapo-transpiration losses from undisturbed cores are being made (1929) by A. A. Young, Assoc. M. Am. Soc. C. E., under the supervision of H. F. Blaney, Assoc. M. Am. Soc. C. E., of the Division of Agricultural Engineering of the U. S. Department of Agriculture, in connection with the Santa Ana River investigation in Southern California, at San Bernardino, with tanks 23 in. in diameter and 40 in. in depth; and at Santa Ana with tanks 23 in. in diameter and 72 in. in depth, all of which are operated by the use of the Mariotte principle.

The effect of applying the water by sprinkling the soil surface has been to increase the evaporation losses two to three times as compared with a stationary water plane fed from below. While these results may not be directly applicable to a comparison between irrigation by sprinkling and by furrows, nevertheless they indicate that, of necessity, the evaporation losses must be much larger for overhead application.

Overhead sprinkling systems for citrus and avocado orchards are installed in Southern California on increasing areas and at costs of as much as $300 per acre, yet sometimes apparently with little regard for the increased evaporation losses. In some cases this form of irrigation has not been successful. Some of the failures must be attributed to the fact that, despite comparatively heavy soils, no additional water supply was provided. With an abundant supply, overhead irrigation can be carried on successfully, provided soils are not too heavy. In such localities the trees appear to be healthy and are bearing abundantly. For steep slopes, this system presents a practical method of overcoming the washing of soils and excessive waste. Tests to establish evaporation losses from sprinkling systems, for different soils, in comparison with furrow irrigation, would be of general interest.

The author has given the mechanical analysis of soils in terms of the percentage of the complete sample. This method may be the most accurate, and, for very coarse materials, the only one, but it does not lend itself to a quick interpretation, except possibly by an experimenter.

Soil designation by the moisture equivalent is becoming a matter of common practice and permits of a ready comparison of different samples. In experiments of the character described, the results of which lend themselves more or less directly to the solution of concrete problems, the writer would suggest that the moisture equivalent and, with undisturbed cores, also the volume weight and porosity of the samples, be given.

The author refers to an experiment* of Professor F. H. King by which it was proved that for dry surface soils the effect of an application of water, which does not produce percolation, is to strengthen capillary flow so that,

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* "The Soil," by F. H. King, 1898, p. 196.

"The surface foot draws upon the deeper soil moisture at a more rapid rate than before, causing a translocation of the lower soil moisture, the deeper soil becoming measurably dryer soon after such a rain than it was before it, while the surface foot is found to contain more water than has fallen on it."

This phenomenon is believed to be responsible for the drying out of bare soils to great depths during periods when rain storms are generally light and even when the maximum storm of the season does not exceed 3 in.

HARRY F. BLANEY,* ASSOC. M. AM. SOC. C. E. (by letter).—In Southern California evaporation losses from soils and river-bed sands are an important factor in water supply studies and could be estimated fairly accurately, provided basic data, such as the author describes, were available. Data of this nature may also prove valuable in estimating surface evaporation from soils immediately after an irrigation. The Division of Agricultural Engineering, Bureau of Public Roads, U. S. Department of Agriculture, in co-operation with the California Department of Public Works, is now making (1929) a study of evaporation losses from moist lands in the Santa Ana River area of Southern California. In this work Mr. Parshall's method of using the Mariotte principle has been extended to tanks 6 ft. in depth, with a water-table maintained at 2 ft. and 4 ft. below the ground surface. The tanks are filled with undisturbed soil and are operated in triplicate.

During 1919 the writer continued the evaporation studies† begun by the late R. B. Sleight, Assoc. M. Am. Soc. C. E., at Denver, Colo., under the direction of Samuel Fortier, M. Am. Soc. C. E., and was very much impressed with the effect of the size of tanks on the depth of evaporation from a water surface. In extending evaporation data obtained from small soil tanks to larger land areas the engineer is confronted with the problem of selecting a proper reduction factor. Very few data are available for accomplishing this. However, factors have been established for extending evaporation data from various sizes of tanks to larger water surfaces. Mr. Sleight‡ concluded that evaporation data from circular tanks at least 2 ft. in depth "may be quite safely extended to large open water surfaces under exactly the same conditions of wind, air temperatures, and relative humidity, by multiplying the evaporation depth from a 2-ft. tank by 0.77, a 4-ft. tank by 0.84, a 6-ft. tank by 0.90, a 9-ft. tank by 0.98 and a 12-ft. tank by 0.99." The writer believes that a somewhat similar relation exists between evaporation from various sizes of tanks containing saturated soils, and that if no better data are available the factors mentioned may be used in estimating the evaporation losses from larger land areas where the water-table is near enough to the surface to saturate the soil.

The data presented by Mr. Parshall in Table 4, for Tank No. 17, August 21 to September 25, are of value in showing the surface evaporation loss after rain storms. Tank No. 17 generally lost a very small amount before the sprinkling started. During this period no water was supplied from the Mariotte tank. About 0.3 to 0.5 in. was applied per week and most of this was evaporated. It is probable that the water applied by sprinkling would not immediately drain down to the water-table, but that at least five days would elapse before equilibrium was again established and, during this time, surface evaporation of the sprinkled water would be taking place. If the quantities applied were small enough, none would move down to the water-table.

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* Irrig. Engr., U. S. Dept. of Agriculture, Los Angeles, Calif.

† *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 303.

‡ "Evaporation from the Surfaces of Water and River-Bed Materials," *Journal of Agricultural Research,* Vol. X, 1917, No. 5, p. 237.

N. W. Cummings,* Esq. (by letter).—The author states that covers were provided to protect the tanks from rain. Presumably, they also prevented solar and sky radiation from reaching the moist surfaces under study because Mr. Parshall states further that a marked reduction in evaporation loss occurs when the covers are in place.

The use of the covers causes the experiments to represent a very important set of conditions, because a vast proportion of moist soil and other earthy material in which engineers are interested is shaded. This fact, however, does not destroy the interest which attaches to a comparison of the shaded and unshaded condition.

When the surface is shaded, nearly all the energy required for supplying the heat of vaporization necessary for converting the water from the liquid to the vapor state, is derived from the air that passes over the evaporating surface. On the other hand, when the surface is unshaded, this atmospheric component of the energy is usually small; it averages about 20% and is in the direction from water to air. This means that air temperature and humidity at the ground level are proportionately vastly more important when the surface is shaded than when it is unshaded.

This is evident from the fact that, in the case of the unshaded surface, atmospheric conditions at the ground level can affect only the smaller components of the energy; they can have no appreciable effect on radiation which is by far the largest component.

It is undoubtedly due, in large measure, to this fact that the depth of evaporation from a standard pan agrees as closely as it does with the loss from a lake subjected to the same sky conditions. The average humidity and air temperature over the lake surface are frequently not the same as over the surface of the water in the pan, yet the depths of evaporation seldom differ by more than 30 per cent.

When the surface is shaded, however, making the atmospheric component of the energy the important factor, there is no reason to expect that results from a small tank will bear anything approaching a definite relation to the loss from a large surface. The atmospheric component of energy depends on both temperature and humidity and as the air passes over an extended surface which is cooler than the air, as a shaded surface is almost sure to be, both temperature and humidity must change in such a direction as to decrease evaporation. Consequently, the loss per unit area might reasonably be expected to decrease very materially, as the area itself increases.

Engineers are confronted, therefore, with the necessity of finding the relation between the loss from a small surface and that from a very large surface under various conditions of wind, air temperature, and humidity when both surfaces are shaded. It is probable that some studies by Jeffreys* may be valuable in connection with this problem.

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* Research Associate, The Scripps Institution of Oceanography, Univ. of California, La Jolla, Calif.

The fact that air temperature at the ground level does not have any marked influence on the largest energy component reaching an unshaded surface may seem inconsistent with the well established relation between air temperature and evaporation from lakes. The apparent contradiction may be easily explained, however, by noting that radiation cannot be controlled by atmospheric conditions at the ground level, even if the temperature may roughly indicate its magnitude.

Carl Rohwer,† Assoc. M. Am. Soc. C. E. (by letter).—The evaporation from moist soils is an important factor in the application of irrigation water to crops and in the determination of the quantity of water stored in the soil, due to rainfall. The measurement of the evaporation presents some difficulty, but the method described by Mr. Parshall permits more accurate control of the ground-water level than is possible by other methods, and materially reduces the work of making observations. As explained by the author, the Mariotte control apparatus is affected by temperature, and care should be used in insulating the equipment as completely as possible.

The principal effect of temperature on the apparatus is in the expansion and contraction of the air in the chamber above the water surface, and in the change in the vapor pressure in the air with the change of temperature. Both effects increase as the temperature increases. The pressure of the air in a closed vessel is directly proportional to the absolute temperature, but the change in vapor pressure with the temperature does not follow so simple a law. The variation, however, has been determined experimentally and shows that there is an appreciable effect on the pressure, due to the change of vapor pressure. The relative magnitude of the two effects of temperature is not important. The only concern is the total effect of the two forces.

When the temperature increases in the Mariotte control apparatus, the pressure in the chamber, *D,* Fig 1, increases and destroys the balance. As a result, water is driven from the water tank. This water usually raises the level of the water in the soil tank slightly, and if the increase in temperature is great enough some of the water will drain out at *K* into *C*. In case the evaporation from the soil surface exceeds the rate at which the water is driven from the water tank, due to the increase in temperature, no error in the record will result. The only effect will be in decreasing the amount of air drawn through the tube, *T,* to maintain the balance. If the temperature drops, the pressure in *D* will decrease and, as pointed out by the author, air should be drawn in through the tube, *T,* to re-establish the balance in the apparatus. Actually, however, the change in temperature is so gradual that the force is not sufficient to break the bubble from the lower end of the tube, *T,* and, as a result, water is drawn from the soil back into the tank. The result of this condition is that the indicated evaporation is too small.

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* Summarized by Humphreys in "Physics of the Air."

† Associate Irrig. Engr., Colorado Experiment Station, Fort Collins, Colo.

The original 1928* records of the observations on the evaporation from saturated soils and river-bed sands show that there was a definite rise and fall of the water in the Mariotte tubes, due to the change in temperature. It was found that Tank No. 17 (Fig. 2), which contained coarse sand and in which the water level was maintained at a depth of 1 ft. below the surface, lost no water during the season. There was, however, a pronounced rise and fall of the water in the Mariotte apparatus which, apparently, bears some relation to the rise and fall of the air temperature. This is shown graphically in Fig. 5, which is a plot of the air temperature at 7:00 A. M. and 7:00 P. M., taken from the meteorological records of the Colorado Experiment Station, and the gain and loss of the water levels in Tank No. 17, in centimeters, for the 12-hour intervals between the 7:00 P. M., 7:00 A. M., and 7:00 P. M. readings for each 24-hour period during the month of August, 1928. The plot indicates, in general, that the water rose in the Mariotte tank when the temperature decreased, and fell when the temperature increased; but the amount of the rise and fall was not proportional to the change in temperature. This is probably due to the fact that since the tanks were carefully insulated, the temperature in the tank did not follow the temperature of the air very closely.

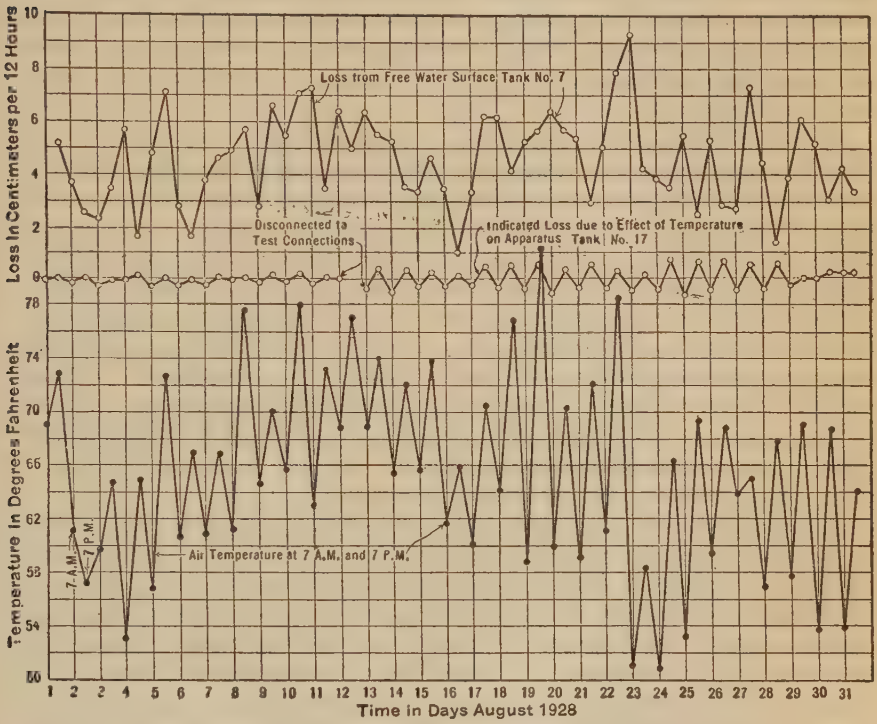

FIG. 5.—EFFECT OF TEMPERATURE ON MARIOTTE CONTROL APPARATUS.

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* Unpublished records of the Irrigation Investigations of the Div. of Agricultural Eng., Bureau of Public Roads, U. S. Dept. of Agriculture, co-operating with the Colorado Agricultural Experiment Station.

For comparison, the record of the evaporation loss from a free water surface under similar conditions, as shown by Tank No. 7 (Fig. 2), is also given. The plot shows that the loss due to evaporation from the free water surface is many times the fluctuation in Tank No. 17 due to the change in temperature. If the comparison had been made on the basis of 24-hour intervals, the difference would have been several times greater because the loss from the free water surface would be nearly doubled and the net change in Tank No. 17 would have been considerably less. The author has shown (Fig. 4) that the evaporation loss from coarse river sand and heavy adobe, Tanks Nos. 14 and 17 (Fig. 2), is very small when the water-table is 12 in. beneath the surface. Temperature probably has an appreciable effect on the indicated evaporation loss from these tanks for a 24-hour period, but by increasing the time interval it is possible to reduce the effect until the desired accuracy is obtained.

Another feature of the Mariotte control apparatus which must be considered is the accuracy with which it controls the level of the water surface in the soil and in the water tanks. No accurate records are available as to the limits within which the apparatus controls the levels in the soil tanks, but the 7:00 A. M. and 7:00 P. M. readings on the hook-gauges in the tanks with the free water surfaces (Tanks Nos. 7 and 8, Fig. 2) give an accurate measure of the limits within which the apparatus permits the water surface to vary. A study of these records for Tanks Nos. 7 and 8 for the 1928 season, shows that there is a variation between 0.02 and 0.04 in. in the elevation of the water surface between the 7:00 A. M. and 7:00 P. M readings, except during periods of rain when any variation is possible, depending on the amount of the rainfall. This, of course, would be corrected by the overflow from the outlet at *K* (Fig. 1), if sufficient time was allowed to elapse.

The Mariotte control apparatus gives reliable results in so far as the control of the water level and the measurement of the evaporation loss are concerned, but there are still some features of the design that need improving because of the difficulty in keeping all the connections air-tight. Even a slight leak permits enough air to leak into *D* (Fig. 1), so that the overflow from *K* will more than fill the receptacle, *B*. Experience with the tanks at Fort Collins, Colo., shows that most of the leaks occur in the connections to the glass gauge tubes. The writer believes that this condition could be corrected by introducing the tube, *T*, into the chamber, *D*, through a packed gland. This would permit using a smaller gauge glass which would be much easier to keep air-tight.

IVAN E. HOUK,* M. AM. SOC. C. E. (by letter).—This paper is valuable to hydrologic engineers for two reasons: First, it presents carefully observed data on the quantities of water evaporated from different soils, with different depths to ground-water; and, second, it describes an automatic method of delivering easily measured quantities of water to soil tanks, as needed to supply evaporation losses, without materially changing the level of the ground-water surface.

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* Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

It is to be regretted that the experiments did not include soil tanks with ground-water surfaces maintained at greater depths than 12 in., especially in the case of the fine sand and loam soils where the 1927 ratios of soil evaporation to free water-surface evaporation for the 12-in. depths varied from 50 to 100 per cent. Depths to ground-water along sandy river channels may vary from zero to several feet. Consequently, engineers engaged in hydrologic studies of such areas need to know the relations between rates of soil evaporation and depths to ground-water for the entire range of depth in which any appreciable quantities of soil moisture are raised to the ground surface by capillary action. Investigations of soil evaporation made thus far seem to indicate that the limiting depth is about 4 ft. in the case of bare sandy or sandy loam soils.

The advisability of using covers to protect the tanks from rain in conducting soil evaporation measurements is questionable. The installation of the covers when the rain begins, their removal when the rain ceases, and the proper adjustment of the evaporation records when some rain unavoidably falls on the soil tanks is probably much more troublesome, as well as more uncertain, than the measurement of total rainfall in a standard rain gauge and the correction of total measured evaporation by total measured precipitation. Moreover, the latter method would furnish data more comparable with actual field evaporation. Nature does not provide covers when the rain begins. The engineer has more need for data on actual rates of soil evaporation immediately following rainfall occurrence than on rates of soil evaporation under watertight covers. For this reason the data in Table 4 are valuable. Conditions of air movement and radiation comparable with those existing under the covers would occur only in the densest jungles of the tropical regions. In most forests the conditions of air movement and radiation at the surface of the ground would be intermediate between those existing under covers and in the open. In this connection the writer would like to know how the average rates of evaporation given in Table 3 were derived. Were they computed for the total times between limiting dates, or for the total times reduced by the intervals during which the covers were in place? The writer would also like to see some photographs showing the general arrangement of the station and typical details of installation.

During the fall of 1926, the U. S. Bureau of Reclamation, in co-operation with the Middle Rio Grande Conservancy District and the U. S. Weather Bureau established a soil evaporation station near Los Griegos, N. Mex., about four miles northwest of Albuquerque. The station was designed for use in studying evaporation from the low undrained bottom-lands and river sandbars along the Middle Rio Grande Valley. It was first equipped with a rain gauge, anemometer, sling psychrometer, maximum and minimum thermometers, two water-surface evaporation pans, two sand pans, and three saltgrass sod pans. Two additional soil pans were added after the station had been operated one year. Fig. 6 shows the arrangement of the equipment and the topography in the vicinity of the station, and Table 7 gives some brief notes regarding the depth and nature of the pans.

Fig. 7 shows the two water-surface evaporation pans; Fig. 8 shows one of the sand pans; and Figs. 8 and 9 also show the three small tubes placed in the soil pans for use in measuring depths to ground-water.

The water-surface evaporation pans were designated Nos. 1 and 2. Pan No. 1, 4 ft. in diameter by 2 ft. deep, was set in the ground to a depth of 21 in., and was kept filled to within 3 in. of the rim. It was more nearly like a floating pan than an ordinary ground pan, since the ground-water table was relatively close to the surface, the depth usually varying from about 8 to 20 in. Pan No. 2 was a standard U. S. Weather Bureau Class A evaporation pan, 4 ft. in diameter by 10 in. deep, mounted on a timber platform, having a surface about 7 in. above the ground.

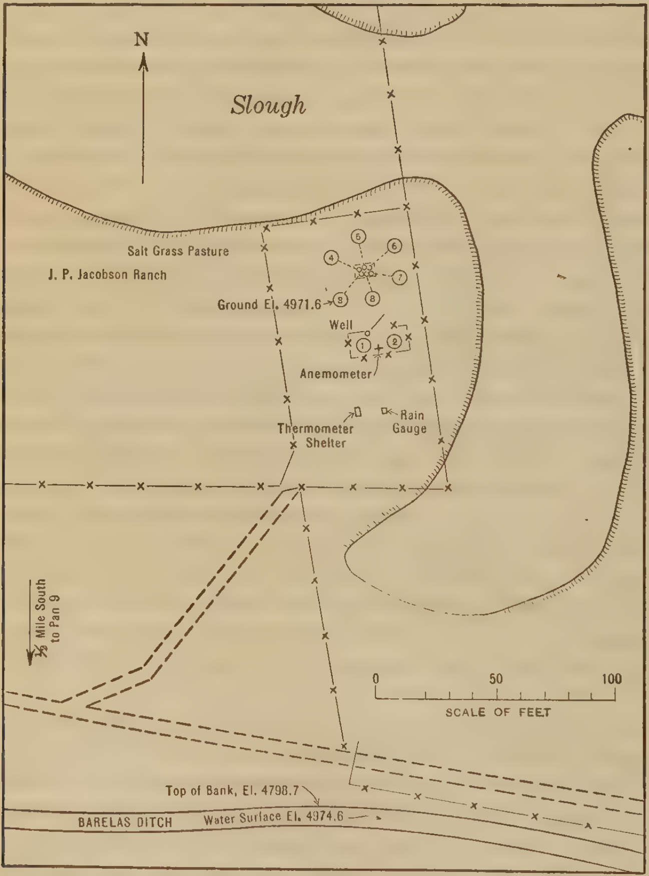
N
Slough
Salt Grass Pasture
J. P. Jacobson Ranch
Ground El. 4971.6
Well
Anemometer
Thermometer Shelter
Rain Gauge
½ Mile South to Pan 9
0
50
100
SCALE OF FEET
Top of Bank, El. 4798.7
BARELAS DITCH
Water Surface El. 4974.6
①
②
③
④
⑤
⑥
⑦
⑧

FIG. 6.—PLAN OF INSTALLATION AND TOPOGRAPHY AT LOS GRIEGOS EVAPORATION STATION.

The plans for the installation of the soil tanks were based on the author's Fort Collins Station, but were modified somewhat so as to secure larger pans and tanks and more substantial pipe connections. Standard 18-gal. hot-water tanks, nominally 12 in. in diameter by 36 in. high, with the usual plumbing connections, were used as supply tanks; and standard farm stock tanks, approximately 4 ft. in diameter, were used as pans. All equipment was carefully tested for water-tightness before installation. The stock tanks were set in the ground, without the use of outside tank containers. The double-tank arrangement is necessary when the soil evaporation is determined by weighing, since then the tanks must be lifted out to be weighed. However, when the Mariotte system is used, about the only advantage of the double tank is that the water-tightness of the outside container can be easily checked at any time by simply removing the inner tank and filling the outer one with water.

TABLE 7.—DESCRIPTION OF PANS SHOWN IN FIG. 6.

Pan No.	Depth, in inches.	Date of installation.	Contents.
1	24	September, 1926	Water.
2	10	September, 1926	Class A, U. S. Weather Bureau.
3	48	September, 1926	Sand, no vegetation.
4	48	September, 1926	Soil, salt-grass sod.
5	36	September, 1926	Soil, salt-grass sod.
6	24	September, 1926	Soil, salt-grass sod.
7	24	September, 1926	Sand, no vegetation.
8	48	September, 1927	Soil, salt-grass sod.
9	36	September, 1927	Soil, tules.

At the Los Griegos Station the water supply pipes were connected directly to the bottoms of the tanks, which were covered with from 6 to 8 in. of coarse gravel, so as to supply water uniformly to the entire bottom of the test soil filling. Pans Nos. 3 and 7 (see Fig. 8) were filled with river wash material composed of a rather fine sand with traces of silt. The material was hauled from a sand-bar on the bank of the Rio Grande and was placed in the pans in water so as to simulate the original manner of deposition. Depths to ground-water were maintained at from 3 to 5 in. in Pan No. 7, and from about 20 to 28 in. in Pan No. 3. Pans Nos. 4, 5, and 6 were filled with the soil excavated to provide holes for the pans. The soil was placed in the pans in thin layers, tamped, and in the same order as excavated. Depths to ground-water were maintained at from 20 to 28 in. in Pan No. 4, from 12 to 19 in. in Pan No. 5, and from 3 to 6 in. in Pan No. 6. Samples of the various soils placed in these three pans were sent for analysis to the Agricultural Experiment Station at State College, N. Mex., and were classified as Gila clay loam. The various strata encountered were described as follows:

Depth below ground surface, in inches.	Material.
0 to 15	Clay loam
15 to 31.5	Fine, sandy loam
31.5 to 42	Clay
42 to 48	Sand

The average results of mechanical analyses of the Gila clay loam soils of the Middle Rio Grande Valley* are given in Table 8.

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* As given in the *Soil Survey Bulletin*, 1912.

A sample of the ground-water used in supplying both soil and water pans was taken August 28, 1926. It was analyzed at the Agricultural Experiment Station at State College, N. Mex., with the following results:

Chemical analysis.	Mineral matter, in parts per 100 000.
Sodium carbonate	23.0
Sodium chloride	8.6
Sodium sulfate	13.2
Total	44.8

The water in Pans Nos. 1 and 2 was renewed frequently, in order to avoid a concentration of alkaline salts which might influence the observed evaporation rate. Some difficulty with the soil tanks was anticipated because of the high content of sodium salts in the water used. However, the salt-grass grew vigorously, especially in Pan No. 6 which received a full supply of water.

During dry periods the evaporation rate at Pan No. 5 was decreased in comparison with that at the other pans, due to the formation of a fine alkaline dust on the surface of the soil. The evaporation rate at Pan No. 3 was also relatively low during such periods, due to the presence of a somewhat similar mulch made up of several inches of loose dry sand. These mulches greatly reduced capillary action.

TABLE 8.—MECHANICAL ANALYSIS OF GILA CLAY LOAM SOILS.

Gradation.	Soil.	Subsoil.	Lower subsoil.
Fine gravel	0.0	0.3	0.3
Coarse sand	1.3	1.3	4.7
Medium sand	2.3	4.0	14.6
Fine sand	15.5	29.8	55.1
Very fine sand	14.7	26.5	17.6
Silt	35.3	17.2	3.7
Clay	30.9	16.2	4.1

For measuring depths to ground-water, three pipes were provided in each soil pan. The pipes terminated a short distance above the saturated layer of gravel at the bottom of the pan. However, the holes at the lower ends gradually opened until finally the elevations of the water-table in the pipes corresponded to the storage-tank pressures. In the case of Pans Nos. 4 and 5 the depths to ground-water measured in the pipes were somewhat less than the actual depths to the water-table in the soil, as determined by additional holes, bored as needed, using care not to penetrate the gravel.

Violent fluctuations in elevation of ground-water surface at Pan No. 7 sometimes occurred due to sudden changes in meteorological conditions. A change of as much as 4 in. sometimes took place within a few hours for no apparent reason, probably due to unnoticeable changes in temperature and barometric pressure. The sudden occurrence of a strong, warm, dry wind would increase the rate of soil evaporation so much that the ground-water would be depleted more rapidly than it could be supplied by the Mariotte flask. Variations in ground-water level also occurred, due to variations in temperature and vacuum at the supply tanks. However, these fluctuations in ground-water level were relatively unimportant as regards the total monthly or annual evaporation losses.

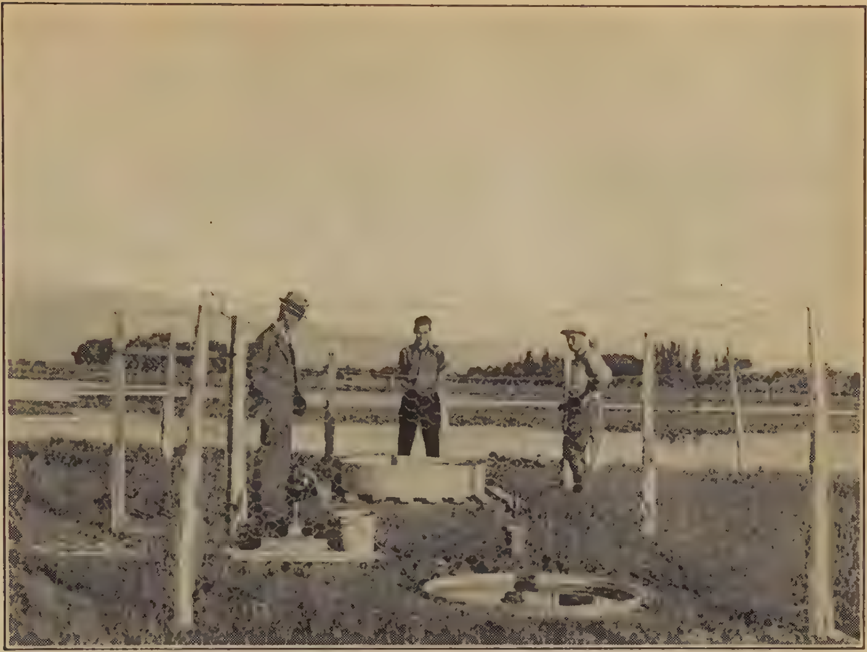

FIG. 7.—PANS NOS. 1 AND 2, FOR MEASURING FREE WATER-SURFACE EVAPORATION, LOS GRIEGOS EVAPORATION STATION.

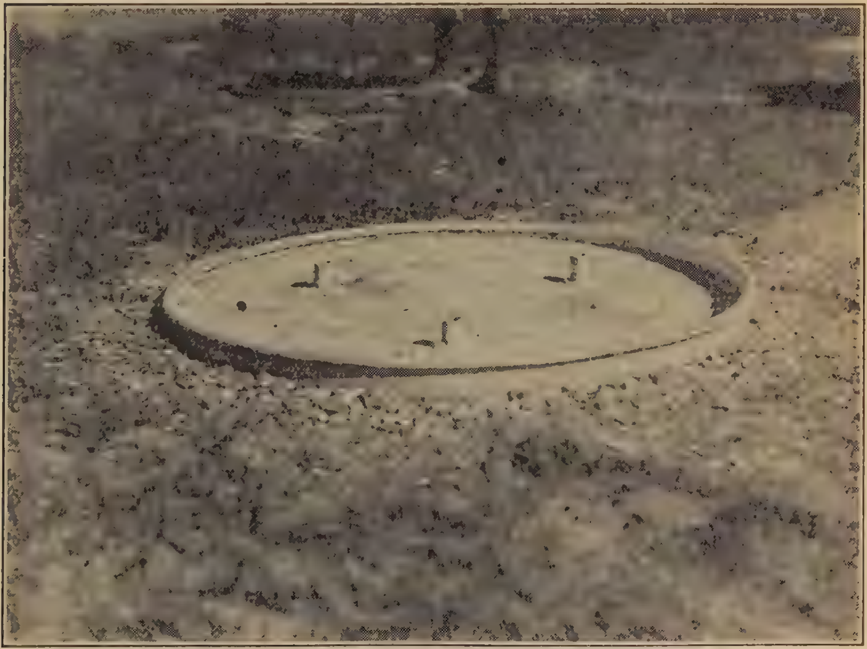

FIG. 8.—PAN NO. 3, LOS GRIEGOS EVAPORATION STATION.

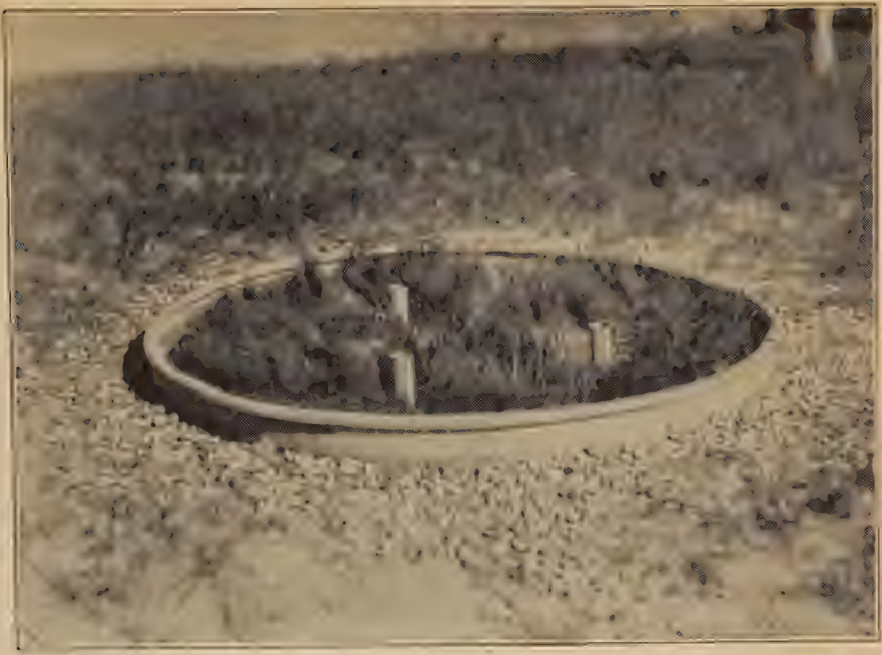

FIG. 9.—PAN NO. 4, LOS GRIEGOS EXPERIMENT STATION.

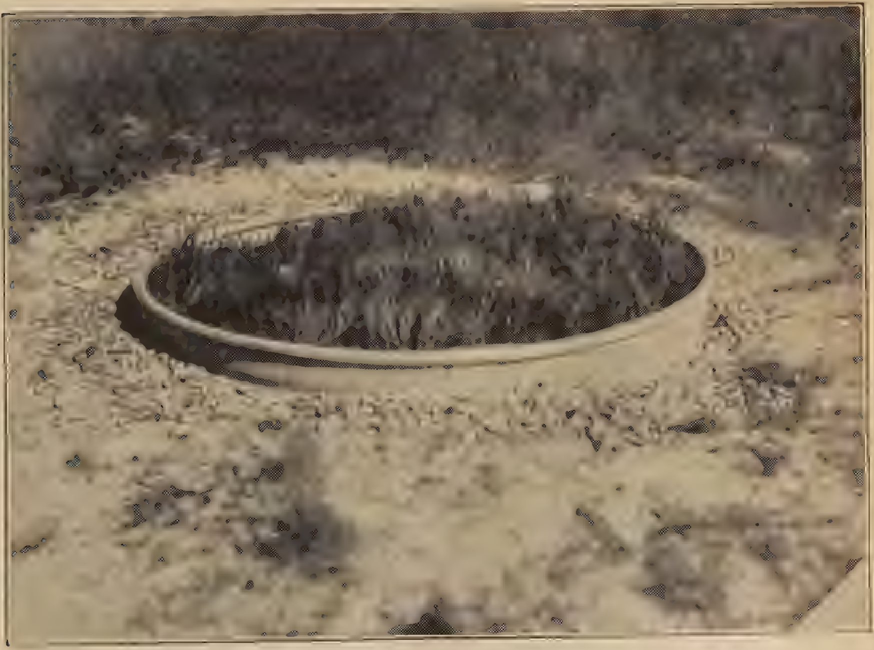

FIG. 10.—PAN NO. 6, LOS GRIEGOS EVAPORATION STATION.

TABLE 9.—METEOROLOGICAL RECORDS AT THE LOS GRIEGOS EVAPORATION STATION, SEPTEMBER, 1926, TO SEPTEMBER, 1928, INCLUSIVE.*

Year.	Month.	Mean temperature, in degrees Fahrenheit.	Total precipitation, in inches.	Mean† wind velocity, in miles per hour.	Mean relative humidity, percentage.	Depth to ground-water at Pan No. 1, in feet.	WATER-SURFACE EVAPORATION.		
							Pan No. 1, in inches.	Pan No. 2, in inches.	Ratio, Column (8) / Column (9) (in percentage).
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1926	Sept...	67.2	1.04	2.7	59	1.2	5.58	7.02	79.5
1926	Oct....	57.3	1.03	2.4	64	1.4	3.77	5.00	75.4
1926	Nov...	44.2	trace	3.5	50	1.3	2.88	4.03	71.5
1926	Dec....	35.4	1.01	2.4	75	1.1	0.90¶	1.27¶	70.8
1927	Jan....	38.9	trace	2.0	64	1.1	1.01	1.45	69.7
1927	Feb....	44.0	0.34	3.6	56	1.2	2.35	3.23	72.8
1927	March.	46.5	0.50	4.7	50	1.2	4.61	6.23	74.0
1927	April...	53.8	0.17	4.0	45	0.9	5.71	8.37	68.2
1927	May...	61.8	trace	4.6	30	1.1	9.22	13.24	69.6
1927	June...	67.9	1.00	3.1	45	1.1	6.83	10.20	67.0
1927	July...	74.7	0.80	2.7	55	1.5	7.80	11.25	69.3
1927	Aug....	71.3	1.62	2.5	60	1.6	6.66	9.12	73.0
1927	Sept...	66.3	1.34	2.8	62	1.3	4.87	6.70	72.7
Averages and totals.......		55.2	7.81	3.19	54.7	1.23	56.61	80.09	70.7
1927	Oct....	54.7	0.19	2.2	48	1.3	3.86	5.54	69.7
1927	Nov...	46.6	0.04	2.5	55	0.9	2.56	3.38	75.7
1927	Dec....	31.5	0.06	3.2	68	1.0	1.07	1.45	73.8
1928	Jan....	33.5	0.00	2.6	33‡	1.1	1.08	2.02	53.5
1928	Feb....	38.2	0.32	3.6	44‡	1.1	1.90	2.90	65.5
1928	March.	47.1	0.06	3.6	31‡	1.1	4.31	6.17	69.9
1928	April...	52.0	0.75	5.1	29§	0.8	6.00	8.62	69.6
1928	May...	61.5	1.38	3.3	39§	0.8	5.34	8.12	65.8
1928	June...	68.5	0.00	3.8	22§	1.5	8.64	12.72	67.9
1928	July...	73.5	1.43	2.4	43§	2.3	7.40	11.00	67.3
1928	Aug....	70.2	2.65	2.6	62§	2.1	7.37	8.39	87.8
1928	Sept...	63.6	0.15	2.3	43‖	2.6	5.33	7.34	72.6
Averages and totals.......		53.4	7.03	3.10	43.1	1.38	54.86	77.65	70.7

* Station ½ mile east of Rio Grande and 4 miles northwest of Albuquerque, elevation 4 970 ft. above mean sea level.

† Anemometer 2 ft. above ground surface.

‡ Mean of 5:00 P. M. readings.

§ Mean of 5:00 P. M. readings for 24 days.

‖ Mean of 5:00 P. M. readings for 21 days.

¶ Pans covered with ice during most of December and up to January 9, 1927.

In the fall of 1927 a study of the first year's records showed the need for a salt-grass sod pan in which the ground-water table could be kept at a somewhat greater depth than was possible with the pans already in use. Consequently, Pan No. 8 was installed, with provisions for maintaining the ground-water surface at a depth of approximately 3 ft. In filling Pan No. 8 the clay layer encountered between depths of 2 ft. 7½ in. and 3 ft. 6 in. was omitted so as to secure data more nearly comparable with those obtained at Pans Nos. 5 and 6. Another new pan (No. 9) was installed at that time. This was planted with tules and the ground-water kept at, or slightly above, the surface of the ground so as to determine the total depths evaporated and transpired

TABLE 10.—EVAPORATION AND TRANSPIRATION LOSSES AT LOS GRIEGOS EVAPORATION STATION.

(All data in feet.)

Year.	Month.	Water-surface evaporation, Pan No. 1.	EVAPORATION FROM SAND AND RIVER WASH MATERIALS.				EVAPORATION AND TRANSPIRATION FROM SALT-GRASS SOD SURFACES.								Tule swamp, Pan No. 9, evaporation and transpiration.
			Pan No. 3.		Pan No. 7.		Pan No. 4.		Pan No. 5.		Pan No. 6.		Pan No. 8.		
			Evaporation.	Depth.*	Evaporation.	Depth.*	Evaporation.	Depth.*	Evaporation.	Depth.*	Evaporation.	Depth.*	Evaporation.	Depth.*	
1926	Oct........	0.314	0.19	2.42	0.26	0.37	0.10	2.07	0.18	1.35	0.29	0.48	......	....	....
1926	Nov........	0.240	0.10	2.32	0.23	0.25	0.01	2.32	0.08	1.15	0.08	0.40	......	...	....
1926	Dec........	0.075	0.12	2.10	0.08	0.20	0.09	2.08	0.09	0.96	0.07	0.20	......	....	....
1927	Jan........	0.084	0.02	2.11	0.12	0.19	0.01	1.84	0.01	1.17	0.03	0.26	......	....	....
1927	Feb........	0.196	0.10	2.33	0.20	0.35	0.04	1.82	0.07	1.24	0.07	0.39	......	....	....
1927	March......	0.384	0.17	2.36	0.40	0.28	0.05	1.67	0.11	1.10	0.14	0.40	......	....	....
1927	April.......	0.476	0.17	2.38	0.47	0.45	0.01	1.67	0.12	1.23	0.26	0.47	......	....	....
1927	May........	0.768	0.16	2.49	0.68	0.41	0.05	2.30	0.26	1.22	0.59	0.37	......	....	....
1927	June.......	0.569	0.16	2.40	0.57	0.36	0.23	2.23	0.38	1.28	0.59	0.41	......	....	....
1927	July........	0.650	0.11	2.50	0.63	0.39	0.29	2.25	0.51	1.27	0.75	0.41	......	....	....
1927	Aug........	0.555	0.20	2.43	0.54	0.34	0.35	2.28	0.61	1.12	0.67	0.40	......	....	....
1927	Sept.	0.406	0.14	2.24	0.40	0.34	0.28	2.30	0.35	1.06	0.49	0.39	......	....	....
Averages and totals ...		4.717	1.64	2.34	4.58	0.33	1.51	2.07	2.77	1.18	4.03	0.38	......	....	....
Percentages...		100.0	34.8	....	97.1	....	32.0	....	58.8	....	85.4	....	......	....	....
1927	Oct........	0.322	0.12	2.38	0.32	0.20	0.16	2.16	0.27	1.34	0.29	0.47	0.04	3.06	0.23
1927	Nov........	0.213	0.08	1.86	0.18	0.42	0.18	2.02	0.10	1.16	0.09	0.45	0.02	3.08	0.15
1927	Dec........	0.089	0.13	1.59	0.11	0.19	0.05	1.89	0.05	1.18	0.05	0.39	0.02	3.10	0.08
1928	Jan........	0.090	0.03	2.10	0.13	0.33	0.01	2.17	0.01	1.10	0.03	0.39	0.01	3.07	0.10
1928	Feb........	0.158	0.08	2.22	0.17	0.23	0.04	2.17	0.03	1.19	0.05	0.40	0.03	3.04	0.14
1928	March... .	0.359	0.09	2.38	0.35	0.31	0.03	2.10	0.05	1.62	0.09	0.49	0.01	3.04	0.30
1928	April.......	0.500	0.10	2.17	0.43	0.31	0.10	2.10	0.16	1.30	0.17	0.40	0.07	3.09	0.43
1928	May........	0.445	0.16	1.78	0.44	0.28	0.22	2.09	0.32	1.30	0.40	0.42	0.13	3.09	0.44
1928	June.......	0.720	0.06	2.18	0.58	0.42	0.18	2.17	0.45	1.41	0.74	0.53	0.02	3.09	0.89
1928	July........	0.617	0.15	2.26	0.57	0.34	0.32	2.33	0.60	1.43	0.84	0.51	0.16	3.09	1.09
1928	Aug........	0.614	0.20	1.90	0.47	0.25	0.41	2.27	0.56	1.41	0.64	0.48	0.28	3.04	0.89
1928	Sept.......	0.444	0.33	2.02	0.37	0.25	0.19	2.25	0.33	1.41	0.48	0.51	0.05	3.07	0.65
Averages and totals ...		4.571	1.53	2.07	4.12	0.29	1.89	2.14	2.93	1.32	3.87	0.45	0.84	3.07	5.39
Percentages...		100.0	33.5	....	90.2	....	41.3	....	64.1	....	84.7	....	18.4	....	117.9

* Average depth of water-table below ground surface.

in tule swamps. This pan was located in a small tule swamp in an old river channel about ½ mile south of the station. The swamp was usually dry in the fall of the year, for a period of from one to two months, due to the lowering of the ground-water level.

Tables 9 and 10* are summaries of the data secured during the first two years of operation. Table 9 gives the monthly meteorological data includ-

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* "Preliminary Report on Middle Rio Grande Investigation, New Mexico," by E. B. Debler, M. Am. Soc. C. E., and C. C. Elder, Assoc. M. Am. Soc. C. E., U. S. Bureau of Reclamation, Denver, Colo., December 15, 1927.

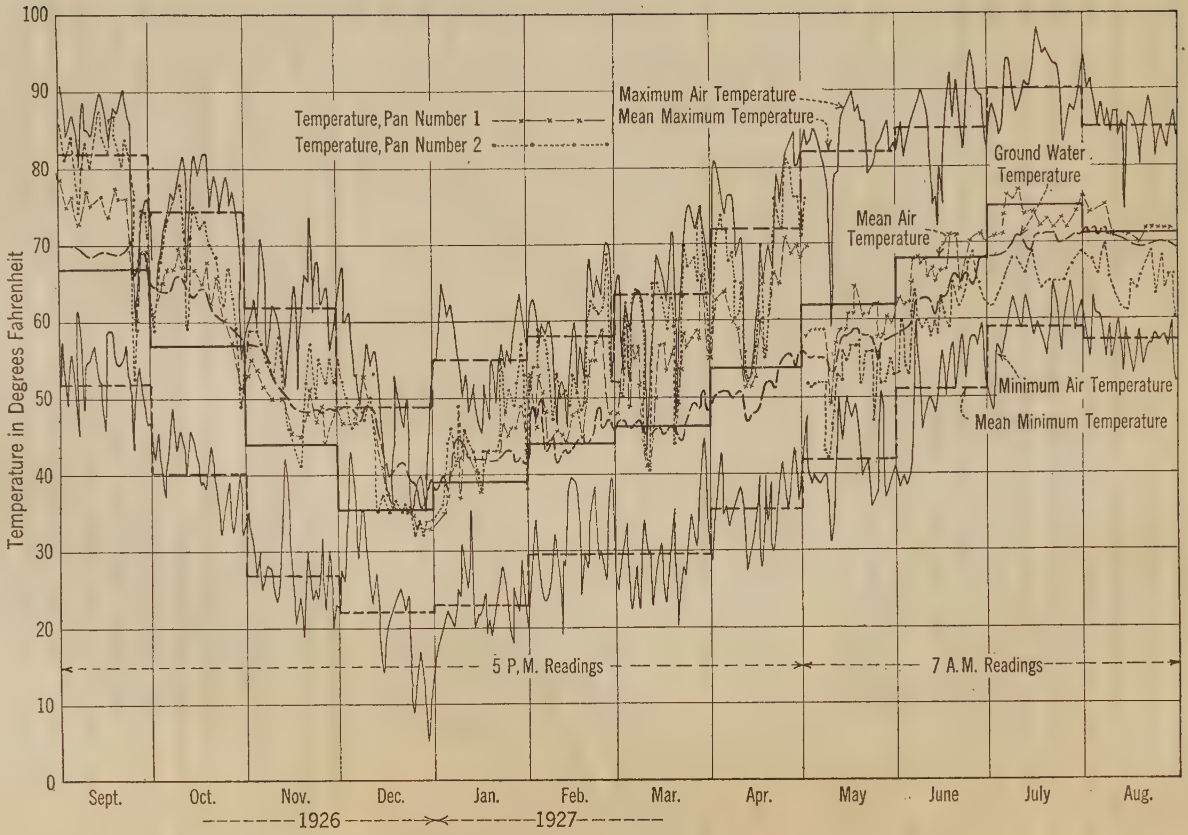

FIG. 11.—AIR AND WATER TEMPERATURES, LOS GRIEGOS EXPERIMENT STATION.

ing the measurements of free water-surface evaporation. It will be noticed that the ratios of monthly ground-pan evaporation (Column (8)) to monthly Class A pan evaporation (Column (9)) vary from about two-thirds to about three-fourths in most cases, and that the corresponding yearly ratios amount to 70.7% in each case (see Column (10)). Table 10 summarizes the data on evaporation and transpiration at the soil pans. Yearly totals and averages, for years ending September 30, are included in both tables, and ratios of soil evaporation to free water-surface evaporation at Pan No. 1 are given in Table 10. The yearly free water-surface evaporation averaged about 56 in. at Pan No. 1, and about 79 in. at Pan No. 2. Annual losses at the salt-grass sod pans varied from 0.84 ft. at Pan No. 8, where the depth to ground-water averaged 3.07 ft., to 4.03 ft. at Pan No. 6 where the average depth to ground-water was about 3 in. Average depths to ground-water at Pans Nos. 4 and 5, as given in Table 10, may be a few inches too low for the reason previously discussed. The yearly loss at Pan No. 9, which was planted with tules, was 5.39 ft.

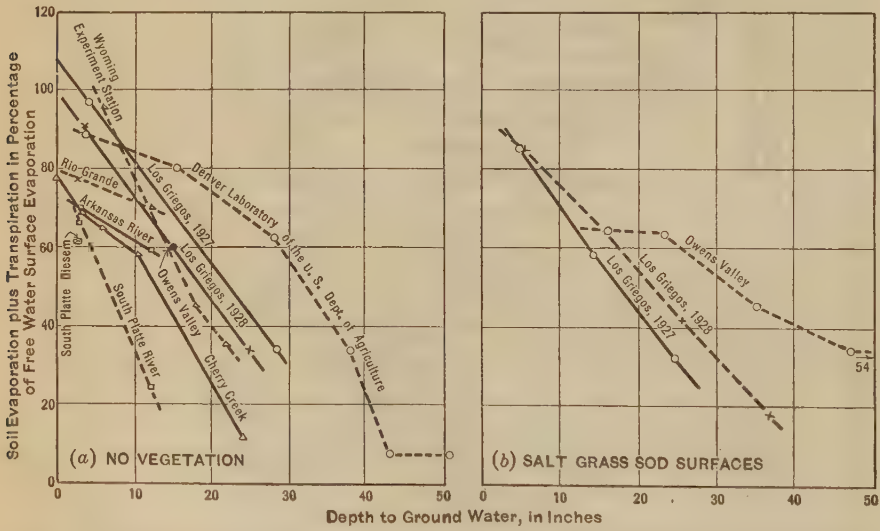

FIG. 12.—RELATION BETWEEN TOTAL ANNUAL EVAPORATION AND DEPTH TO GROUND-WATER.

Fig. 11 shows the daily meteorological records, including temperatures of the ground-water and of water in Pans Nos. 1 and 2, during the year ending August 31, 1927. Daily readings were taken at about 7:00 A. M. during the months of May, June, July, and August, and at about 5:00 P. M. during the remainder of the year. The ground-water temperatures did not vary greatly from the mean monthly air temperatures during any part of the year, although they were slightly lower than the air temperatures during the summer when the measurements were made in the morning, and slightly higher than the air temperatures during the remainder of the year when the observations were made late in the afternoon. Water temperatures in the evaporation pans were nearly as great as the maximum air temperatures when the observations were made at 5:00 P. M. and slightly lower than the mean air temperatures when the readings were made at 7:00 A. M. Water temperatures in Pan No. 1 were a few degrees lower than those in Pan No. 2, as would be expected.

Fig. 12(*a*) shows the annual soil evaporation at Pans Nos. 3 and 7, in percentage of free water-surface evaporation, plotted against the average depths to ground-water. For comparative purposes similar data obtained at the Wyoming Agricultural Experiment Station,* at the Denver Laboratory of the U. S. Department of Agriculture,† and in Owen's Valley, California,‡ have been added on the diagram. The data presented by the author have not been added since they are similarly shown in Fig. 4. Fig. 12 (*a*) would indicate that but little, if any, evaporation occurs from the surfaces of bare, sandy soils when the depth to ground-water exceeds about 4 ft.

Fig. 12 (*b*) shows the annual soil evaporation and transpiration at the salt-grass sod tanks, Pans Nos. 4, 5, and 6, the Owen's Valley data being added for comparison. In this case the results of the Owen's Valley investigations constitute the only similar data available. It will be noticed that the ratios of soil evaporation and transpiration to free water-surface evaporation were appreciably lower at the Los Griegos Station than at the Owen's Valley Station. The Los Griegos records were also more consistent as regards relation of evaporation losses to average depths to ground-water. The Los Griegos experiments seem to indicate that the loss of soil moisture by evaporation and transpiration, from the Gila clay loam covered with salt-grass sod, practically ceases when the depth to ground-water reaches about 4 ft.

RALPH L. PARSHALL,§ ASSOC. M. AM. SOC. C. E. (by letter).—The writer is gratified in the general interest shown in the subject of evaporation from moist soils, by the several discussions. Because of water supply problems, especially in the West, now being studied in considerable detail, the matter of evaporation and transpiration losses from extended areas of both irrigated and non-irrigated lands requires basic information in the solution of these important problems.

Numerous studies of evaporation losses have been made by previous investigators. The present apparent lack of data covering a wide range of soils, depths to water-table, and various kinds of plants, offers the opportunity of more extended and detailed study of this important research problem, and Mr. Fortier, in his very able discussion, points out the immediate need of a more complete investigation of this nature. The question of water supplies is becoming more and more important and the element of evapo-transpiration losses, bearing on the problem, likewise is becoming more urgent. Present investigations of this subject are still confined to tanks of relatively small dimensions which necessitate the application of correction factors for extended areas.

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* *Bulletin 52,* Wyoming Agricultural Experiment Station.

† "Evaporation from the Surfaces of Water and River Bed Materials," by R. B. Sleight, *Journal of Agricultural Research,* Vol. X, No. 5, July 30, 1917.

‡ "An Intensive Study of the Water Resources of a Part of Owen's Valley, California," by Charles H. Lee, U. S. Geological Survey, *Water Supply Paper 294.*

§ Irrig. Engr., U. S. Dept. of Agriculture, Fort Collins, Colo.

The want of such factors as mentioned by Mr. Blaney is fully appreciated. The application of such correction factors for evapo-transpiration losses, based on the evaporation from a free water surface, or from a saturated soil surface, can not be reasonably approximated. The vegetative draft on the soil moisture is much more than is commonly supposed and, therefore, until more extensive studies are made on a large scale, covering a rather wide range of conditions, the engineer will be obliged to apply very uncertain corrections in summing up the losses due to evaporation and plant use. The investigation by Messrs. Blaney and Young in California is without doubt the most extensive and detailed study yet attempted, and from this work will come a substantial advancement in the knowledge of this important question.

Mr. Sonderegger's statement that the losses from the fine river-bed sands were from disturbed material is true. However, as these sands were originally laid down by the deposition from the flood flows of the river, it may be fairly concluded that, in the case of the experimental set-up of these materials, the density or compaction was essentially the same as in the original bed. The practical engineering use of the data resulting from experimental studies must of necessity be applied to a very wide range of soil conditions. To assume that the results of an investigation of a single soil condition are applicable to the wide variation found over an extended area, would therefore seem to be rather doubtful. The writer is somewhat inclined to question whether any marked difference in the rate of evaporation from a cored sample or otherwise would be evident in an experimental set-up.

The use of sprinkling in the application of water to the soil, is believed to be uneconomical from two standpoints: First, if the application is moderate, the cooling of the ground surface and the temporary linking up of the capillary movement have been shown to deplete the soil moisture; and, second, the fact that the spray vastly increases the evaporating area of the water surface results in a much greater loss than if the water was applied as a stream in contact with the ground. As pointed out by Mr. Sonderegger, the classification of soil by the moisture equivalent percentage, from the standpoint of evaporating medium, is no doubt much more satisfactory than the simple classification of a mechanical analysis. It seems reasonable to suppose that possibly the rate of capillary movement in a soil column might be a better index than either the mechanical or the moisture equivalent. The apparent evaporation loss from soil surfaces where the water-table is at some depth, is, in effect, a function of the capillary movement of the moisture to the surface where it is dissipated as evaporation. The loss can only be as rapid as the capillary movement of the soil moisture from the water-table to the surface.

The proposal to place covers over the experimental soil tanks, as a means of protection against rain, may be considered questionable. Experience indicates that where rain falls on the exposed area, an element of uncertainty in the resulting records becomes apparent, because of the inability to determine the amount of the correction to be applied due to the precipitation. Splash from a dashing shower, wide rims on the tanks, and the inherent inaccuracy of measuring the rainfall in a standard rain gauge, all result in a doubtful cor-

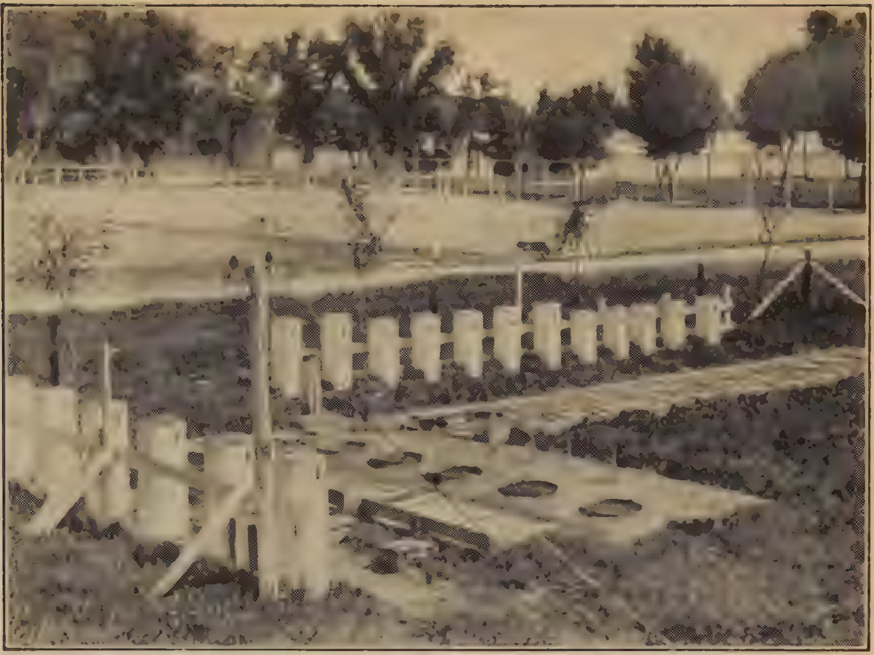

Fig. 13.—General View of Arrangement of Soil Tanks and Meteorological Equipment.

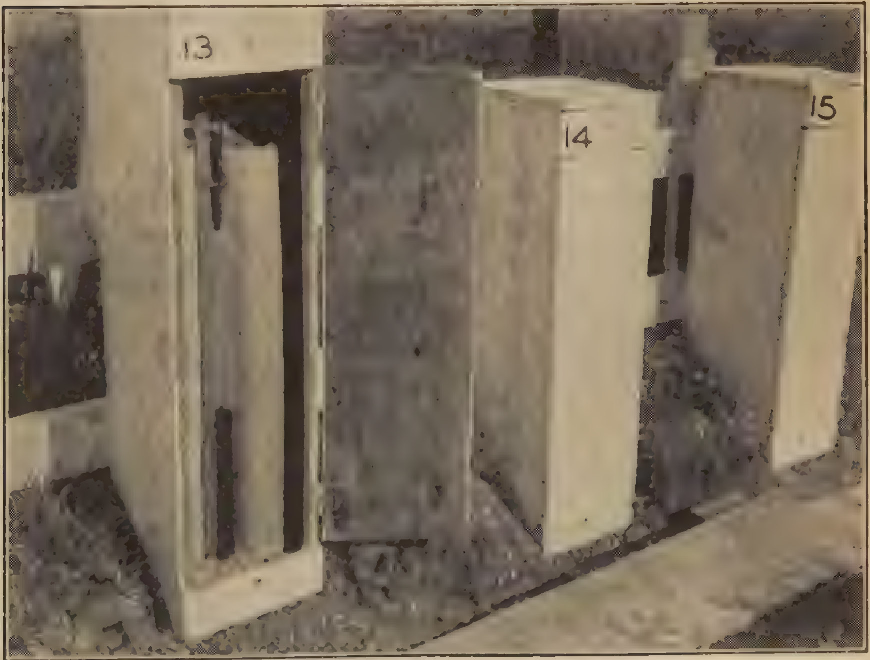

Fig. 14.—Housing for Mariotte Tube Regulation.

rection factor; also, the soil surfaces, when disturbed by a wetting, immediately assume a different evaporating medium. To eliminate these uncertainties, covers were provided. The rate of evaporation with the covers on resulted in a marked decrease in the rate of loss. Ordinarily, these covers were on only for a short period to protect against a passing shower. Inasmuch as all tanks were covered for the same period, the relative losses may be consistently compared.

The discussion by Mr. Cummings brings forward the interesting relation of the atmospheric component of energy to the phenomena of evaporation. Energy is expended in the dissipation of water by evaporation and when the surface of the evaporating medium is shaded, the incoming solar energy is reduced. The experiments in question were not in sufficient detail to draw conclusions as to this effect. The covers were partly opened at the ends and at one side, thus permitting some circulation of air currents. However, as the sun was shaded by clouds at the time the covers were in place, no direct relation of shaded and unshaded effects is possible. It is assumed that the most important factors tending to reduce the rate of evaporation when the covers are in place, are the increased humidity due to the rain and the protection against air movement.

Mr. Houk's discussion presents valuable data, especially for depths of soils exceeding those reported by the writer. At the time of the set-up of the apparatus to study the evaporation loss from moist soil surfaces, it was the intention to determine the relative losses for shallow depths of water-table for various types of fine sands and other soils. In 1913 a series of experiments were conducted by Mr. V. M. Cone to determine the rate of capillary rise in soil columns about 1 in. in diameter. It was found that, for a fine sandy loam soil, taken at Eads, Colo., more than 100 days were required for the moisture to reach a height of 58 in. For the river-bed sand, as used by Mr. Houk, the writer would be inclined to assume that the loss would approach zero for a water-table much less than 4 ft. For the medium fine sand (Fig. 4), it seems evident that no loss would occur at about 20 in.

The data presented in Table 3 give the average daily loss per 24 hours for the period indicated. The first tabulation of the results of this investigation gave the average loss per 24 hours for each week and, in compiling these data, corrections were made where the covers were in place for several hours. For the most part the covers were put on just long enough to protect the tanks during a shower and for these short periods no attempt was made to apply a correction.

Fig. 13 is the general arrangement of soil tanks with housing over the Mariotte tube regulators. (See, also, Fig. 2.) Fig. 14 shows Tank No. 13 with the exposed glass tube and adjacent fixed meter stick used in determining the evaporation loss. Previous to the use of the Mariotte principle as applied to maintaining and regulating the water supply for an evaporation tank, an arrangement very similar to the apparatus described by the writer was used by O. V. P. Stout, M. Am. Soc. C. E., and Carl Rohwer, Assoc. M. Am. Soc. C. E., in the study of seepage losses from canals in California.

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Paper No. 1749

PROBLEMS CONCERNING ELASTIC STABILITY IN STRUCTURES*

By S. Timoshenko,† Esq.

With Discussion by Messrs. H. M. Westergaard, William Hovgaard, A. Nadai, and S. Timoshenko.

Synopsis

Sometimes the strength of a structure does not depend on the ultimate strength of the material used, but on the elastic stability of the members. Such problems become of great practical importance in steel structures where materials of high strength are used. Problems of this kind, involving buckling of built-up columns, buckling of a compressed top chord of a half-through span, and stability of webs in compressed members and girders, are considered.

I.—Introduction

The simplest case in which the strength of a structure does not depend on the ultimate strength of the material, but on the elastic stability of the members of the structure, is a strut or a column under the action of a compressive force. If the length of the strut is large in comparison with its cross-sectional dimensions, failure will occur as a result not of a high compressive stress but of elastic instability. The strut may buckle sidewise at a stress less than the yield point of the material.

In steel structures, materials of high strength are used and, as a consequence, the transverse dimensions of members are likely to become small in comparison with the length. With the modern tendency to increase the yield point of structural steels the field of application of theoretical solutions, based on the assumption of a perfectly elastic material, increases and these solutions are, therefore, of practical interest. Several solutions of this kind will be given and their application to such problems as stability of built-up columns, stability of I-beams, and stability of webs of girders, will be considered.

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* Presented at the meeting of the Structural Division, Columbus, Ohio, October 13, 1927.

† Prof. of Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

II.—STABILITY OF BUILT-UP COLUMNS

A solid column with hinged ends will be taken as a fundamental case and the critical loads, P_{cr}, for different built-up columns will be compared with the known critical load for this case, which is:

$$P_{cr} = \frac{E\,I\,\pi^2}{l^2} \quad\text{(1)}$$

in which, $E\,I$ is the flexural rigidity of the column in the plane of buckling, and l is the length of the column.

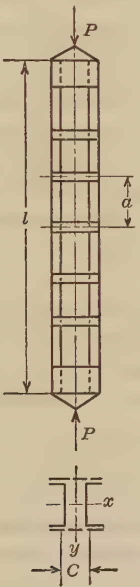

FIG. 1.

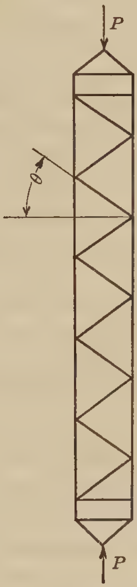

FIG. 2.

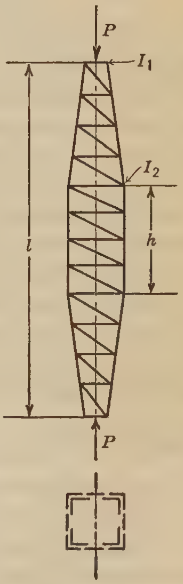

FIG. 3.

Critical loads for built-up columns are always less than those for solid columns having the same value of $\frac{E\,I}{l^2}$ and depend on such details as lattice-bars and spacing plates, or battens. The value of critical loads for such columns can always be represented by the equation:

$$P_{cr} = \alpha\,\frac{E\,I\,\pi^2}{l^2} \quad\text{(2)}$$

in which, α is a factor less than unity.

Take, for instance, the case shown in Fig. 1. The factor, α, in Equation (2) for this case can be represented in the following form.*

$$\alpha = \frac{1}{1 + \frac{E I \pi^2}{l^2}\left(\frac{a c}{12 E I_2} + \frac{a^2}{24 E I_1}\right)} \quad \text{.....(3)}$$

in which,

a = distance between the axes of battens.

c = distance between centers of gravity of channels.

I = moment of inertia of the cross-section about the y-axis.

I_1 = moment of inertia of the cross-section of a channel about the axis through its center of gravity and parallel to the y-axis.

I_2 = moment of inertia of the cross-section of a batten about the axis through its center of gravity and perpendicular to the axis of the column.

It is seen that the strength of the column against buckling depends not only on the value of $\frac{E I}{l^2}$, but also on the rigidity of the channels and battens of which it is made.

Numerical Example.—For a cross-section consisting of the 15-lb., 10-in. channels, let it be assumed that l = 30 ft., a = 36 in., c = 7.3 in., the thickness of battens = $\frac{3}{8}$ in., and the width of battens = 9 in. Then, for the cross-section, calculated as a solid column, I = 123 in.^{4}, while I_1 = 2.3 in.^{4}, and I_2 = 45.6 in.^{4}. Substituting in Equation (3), $\alpha = 0.82$. Hence, in this case, the ultimate strength of the built-up column is 0.82 of the critical load of the corresponding solid column.†

If the proportions of a built-up column are such that the compressive stress calculated from Equation (2) is beyond the yield point of the material, the factor, α, will be greater than that given by Equation (3). Hence, the difference in strength between the two types of columns will diminish with a decrease of the ratio, $\frac{l}{r}$. This conclusion can also be deduced from Equation (3). In order to obtain P_{cr} analytically when the stress as calculated by Equation (2) is beyond the yield point of the material, the constant quantities, $E I$ and $E I_1$, in Equation (3), because of the decrease in modulus of elasticity above the yield point, must be replaced by variable quantities diminishing with the increase in compressive stress. At the same time, the rigidity, $E I_2$, of the battens, which are not subjected to compression, remains equal to unity. This means that the value of P_{cr}, calculated from Equation (3), for the cases of short columns, will always be on the safe side.

In the case of a latticed column (Fig. 2), the values of the factor, α, in Equation (2) can be calculated from the equation:

$$\alpha = \frac{1}{\frac{E I \pi^2}{l^2} \; \frac{1}{E \sin\theta \cos^2\theta\, A}} \quad \text{.....(4)}$$

in which, A is the cross-sectional area of a lattice-bar.

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* For the derivation of this equation see "Applied Elasticity," by S. Timoshenko and J. M. Lessells, pp. 180–182.

† A central application of the compressive force, P, is always assumed.

The latticing is usually so proportioned that the factor, α, differs little from unity. Then, the formula for solid columns can be used.

Consider now a square column shown in Fig. 3. The middle part (of the length, h) has a constant moment of inertia, I, decreasing toward the ends, where it is I_1. Assuming that the latticing is proportioned in such a manner that the column can be considered as solid and of variable cross-section, the critical value of compressive force will be found from the equation:

$$P_{cr} = \beta \frac{E\,I}{l^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which, β is a numerical factor, the value of which depends on the ratios, $\frac{h}{l}$ and $\frac{I_1}{I}$. Evidently, the value of β must approach π^2 when the ratios, $\frac{h}{l}$ or $\frac{I_1}{I}$, approach unity because then the column becomes almost uniform in cross-section. Some values of the factor, β, are given in Table 1.*

TABLE 1.—VALUES OF THE FACTOR, β, IN EQUATION (5).

Values of $\frac{I_1}{I}$.	VALUES OF $\frac{h}{l}$.					
	0.0.	0.2.	0.4.	0.6.	0.8.	1.0.
0.0001	1.000	2.972	4.754	7.658	9.622	π^2
0.2	6.374	7.488	8.611	9.443	9.814	π^2
0.4	7.614	8.420	9.149	9.634	9.838	π^2
0.6	8.512	9.038	9.477	9.736	9.853	π^2
0.8	9.243	9.499	9.699	9.817	9.859	π^2
1.0	π^2	π^2	π^2	π^2	π^2	π^2

III.—BUCKLING OF STRUTS HAVING LATERAL ELASTIC SUPPORTS

This type of problem is encountered, for instance, in considering sidewise buckling of the compressed top chord of a half-through truss span. As a simple case, assume that a strut of uniform cross-section, when buckled under the action of compressive forces, P (Fig. 4), encounters a resistance produced by an elastic medium such that the lateral reactions continuously distributed along the strut are proportional to the corresponding deflections. Because of these lateral reactions the critical load will be greater than that given by Equation (1). The magnitude of this load can be found from a consideration of the potential energy of the system.† The critical value of the compressive force, P, is that at which the work done during sidewise buckling by the compressive forces is equal to the potential energy of bending of the strut, together with the energy of deformation of the elastic medium. For the potential energy of bending of the strut the following equation applies:

$$V_1 = \frac{E\,I}{2} \int_0^l \left(\frac{d^2\,y}{d\,x^2}\right)^2 d\,x \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

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* This table was calculated by Prof. A. Dinnik, Polytechnical Inst., Ekaterinoslaw (Russia).

† For discussion of this energy method, see "Applied Elasticity," p. 169.

In considering the potential energy of the elastic medium, a factor, K, called the modulus of the medium, is taken such that the product, $K\,y$, represents the reaction of the medium per unit length of the strut when the deflection is equal to y. Then the work done by this reaction during deflection will be $-\frac{1}{2}K\,y^2$ and, consequently, the potential energy of the deformed elastic medium will be:

$$V_2 = \frac{K}{2}\int_0^l y^2\,d\,x \qquad (7)$$

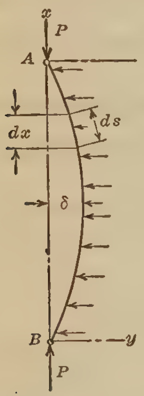

FIG. 4.

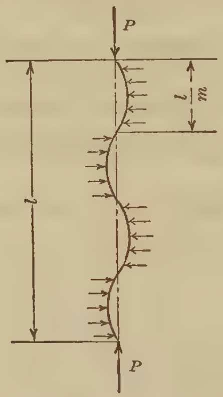

FIG. 5.

The work done by the compressive forces during buckling will be obtained by multiplying P by the displacement of the end, A, with respect to B, which occurs during sidewise buckling of the strut. Taking into consideration the fact that the difference between the element, $d\,s$, of the deflection curve and its vertical projection, $d\,x$ $\left(\text{since } \frac{d\,y}{d\,x} \text{ is small}\right)$, is equal to:

$$d\,s - d\,x = d\,x\sqrt{1 + \left(\frac{d\,y}{d\,x}\right)^2} - 1 = \frac{1}{2}\left(\frac{d\,y}{d\,x}\right)^2 d\,x$$

the work, T, produced by the compressive forces during buckling will be:

$$T = \frac{P}{2}\int_0^l \left(\frac{d\,y}{d\,x}\right)^2 d\,x \qquad (8)$$

Now, from Equations (6), (7), and (8), the condition for the critical load will be:

$$\frac{1}{2}P\int_0^l \left(\frac{d\,y}{d\,x}\right)^2 = \frac{1}{2}E\,I\int_0^l \left(\frac{d^2\,y}{d\,x^2}\right)^2 d\,x + \frac{1}{2}K\int_0^l y^2\,d\,x \qquad (9)$$

If the elastic medium has a small rigidity so that the factor, K, is small, the deflection curve will be the same as in the case of a strut without lateral support. Therefore, it can be represented by:

$$y = c\sin\frac{\pi\,x}{l} \qquad (10)$$

Substituting this in Equation (9),

$$P_{cr} = \frac{E\,I\,\pi^2}{l^2}\left(1 + \frac{K\,l^4}{E\,I\,\pi^4}\right) \qquad (11)$$

The second term in the brackets represents the effect of the elastic medium on the critical load.

In the case of a medium of greater rigidity, the deflection curve of the buckled strut has a more complicated shape consisting of several waves (Fig. 5). With an increase in rigidity of the medium, the number of waves of the deflection curve increases, but each wave is a simple sine wave and for calculation of critical loads Equation (11) can be used. To do this it is only necessary to substitute in this equation, instead of the length, l, $\frac{l}{m}$, in which, m denotes the number of waves. Then:

$$P_{cr} = \frac{E\,I\,\pi^2}{l^2}\left(m^2 + \frac{K\,l^4}{E\,I\,\pi^4\,m^2}\right) \qquad (12)$$

In each case, m must be chosen in such a manner as to make the sum in the bracket a minimum. This number, which depends on the rigidity of the medium, increases with the increase of the quantity, K, and can be determined in the following manner: Let K be the rigidity at which the number of waves in the deflection curve of the strut changes from m to $m + 1$. Then, the sum in the brackets of Equation (12) will remain unchanged by replacing m by $(m + 1)$, and, consequently,

$$m^2 + \frac{K\,l^4}{E\,I\,\pi^4\,m^2} = (m + 1)^2 + \frac{K\,l^4}{E\,I\,\pi^4\,(m + 1)^2} \qquad (13)$$

from which,

$$m^2\,(m + 1)^2 = \frac{K\,l^4}{E\,I\,\pi^4} \qquad (14)$$

In this manner for each value of m the limiting value for the rigidity, K, can be calculated. For a value of K slightly less than this calculated value, the deflection curve will have m waves, while for a value of K slightly greater the deflection curve will have $m + 1$ waves.

If the rigidity, K, or the length, l, are very large, the number, m, will also be large and, neglecting unity in comparison with m in Equation (14),

$$m^4 = \frac{K\,l^4}{E\,I\,\pi^4}$$

Substituting this in Equation (12),

$$P_{cr} = \frac{2\,E\,I\,\pi^2\,m^2}{l^2} \qquad (15)$$

The critical load in this case is thus twice as great as in the case of a strut without lateral support and having a length equal to,

$$\frac{l}{m} = \sqrt[4]{\frac{E\,I\,\pi^4}{K}} \qquad (16)$$

IV.—BUCKLING OF COMPRESSED TOP CHORD OF A HALF-THROUGH TRUSS SPAN

To determine the critical compressive force for the top chord of a half-through truss span is much more complicated because the compressive forces are distributed along the chord, the section of the chord is not constant, and instead of a continuous elastic medium there are lateral elastic supports represented by vertical truss members. Consider the simplest case of a through-truss span of a constant height, h (Fig. 6), and assume that the top chord has a uniform cross-section and that all the vertical truss members are of the same section except the end verticals which are so rigid that the ends of the top chord do not move laterally during buckling of the chord.

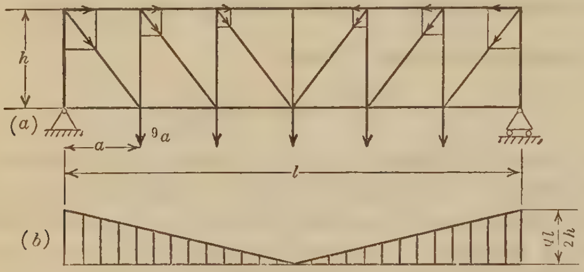

FIG. 6.

Assuming that the truss is subjected to the action of a uniformly distributed load of intensity, q, the tensile forces in the diagonals and their horizontal components, producing compression in the top chord, will be proportional to the distances from the middle of the span. Replacing the concentrated compressive forces acting on the top chord by equivalent continuously distributed forces, the distribution of the compressive forces will be as shown by the shaded area in Fig. 6 (b). The maximum compression will be at the middle of the chord and will be equal to $\frac{q\,l^2}{8\,h}$. The critical value of this compressive force corresponding to the beginning of sidewise buckling of the top chord can be found in the same manner as in the simple case of a strut in an elastic medium, already discussed. It is only necessary to replace the concentrated elastic reactions of the verticals, opposing the lateral deflection of the compressed chord, by continuously distributed reactions of an equivalent elastic medium. The modulus, K, of the equivalent medium will be:

$$K = \frac{Q}{a} \quad \text{.............................} \quad (17)$$

in which, Q is the lateral force which, if applied to the vertical member at the top chord panel point (neglecting for the moment the lateral resistance of the chord) will produce a lateral deflection of the vertical member equal to unity; and a is the distance between the verticals.

Assuming, for instance, that the lower ends of the verticals are absolutely fixed and putting $E\,I_1$ equal to the flexural rigidity of the verticals in a lateral direction, the force, Q, will be obtained from the expression,*

$$\frac{Q\,h^3}{3\,E\,I_1} = 1$$

from which,

$$Q = \frac{3\,E\,I_1}{h^3} \quad \text{(18)}$$

Calculating K from Equations (17) and (18), and using an energy equation similar to Equation (9), it will be found (see Appendix) that the critical value of the maximum compressive force in the top chord can be represented by:

$$\left(\frac{q\,l^2}{8\,h}\right)_{cr} = \gamma\,\frac{E\,I\,\pi^2}{l^2} \quad \text{(19)}$$

in which, γ is a numerical factor, greater than unity, and depending on the modulus, K, the lateral flexural rigidity, $E\,I$, of the top chord, and the span, l.

TABLE 2.—NUMERICAL VALUES OF THE FACTOR, γ, IN EQUATION (19), ENDS OF TOP CHORD FIXED LATERALLY.

$\frac{K\,l^4}{16\,E\,I}$ =	0	5	10	15	22.8	56.5	100	163	200	300	500	1 000
γ =	2.06	3.63	5.10	6.37	7.58	9.51	11.9	14.9	16.5	19.8	24.0	33.0

By using Table 2 the factor, γ, can be found for a given value of $\frac{K\,l^4}{16\,E\,I}$. Then, the critical compressive force in the chord will be the same as for a simple strut, the length of which is equal to $\frac{l}{\sqrt{\gamma}}$. For $K = 0$ the case is that of a strut without lateral supports and loaded as shown in Fig. 6 (*b*). The critical compressive force is then about twice as great ($\gamma = 2.07$) as for a strut compressed by forces applied at the ends.

This method can be applied also in the case when end verticals are of the same rigidity as the intermediate verticals and the ends of the top chord are free to move laterally during buckling. Equation (19) can be used also in this case, but the numerical values of the factor, γ, are, however, somewhat different (Table 3).

TABLE 3.—NUMERICAL VALUES OF THE FACTOR, γ, IN EQUATION (19), ENDS OF TOP CHORD FREE TO MOVE LATERALLY.

$\frac{K\,l^4}{16\,E\,I}$ =	0	1	3	5	10	15	20	50	100	150	200	300	500	1 000
γ =	0	0.203	0.610	1.015	2.02	3.03	3.40	5.38	8.64	13.05	14.70	17.98	23.6	32.3

---

* The bending of the cross-beam of the floor structure and the effect of the compressive force in the verticals on the deflection are neglected in this equation.

With an increase in the quantity, $\frac{K\,l^4}{16\,E\,I}$, the values of γ approach those given in Table 2 for rigid end verticals. This result is easily understood since for the compressed top chord the deflection curve during lateral buckling may have several waves as in the case of a strut in an elastic medium (Fig. 5). The number of waves increases with the rigidity, K; hence when this number is large, the critical load is no longer affected substantially by a variation in the end conditions.

V.—Lateral Buckling of I-Beams

It is known that, in the absence of lateral supports, an I-beam bent in the plane of the web may prove to be insufficiently stable. If the loads are increased beyond a certain critical limit, it buckles sidewise and collapses. The critical load can be found by the energy method; for a beam on two supports it always can be represented in the form:

$$P_{cr} = \frac{n\sqrt{B_2\,C}}{l^2} \quad \text{......................} (20)$$

in which,

$B_2 = E\,I_2$ = the flexural rigidity of the beam in the direction perpendicular to the web.

C = the torsional rigidity of the I-beam in twisting.

l = the span.

n = a numerical factor depending on the distribution of load, on the manner of fastening the ends of the beam, and on the magnitude of the ratio, $\frac{C\,l^2}{B_2\,h^2}$, h being the depth of the beam.

Tables of the factor, n, for various cases, and the application of Equation (20) have been discussed previously.* In Fig. 7,† beams of very narrow rectangular cross-section, instead of I-section, are taken. For such a beam supported at the ends, Fig. 7 (a), the critical load, P_{cr}, at which sidewise buckling begins, is:

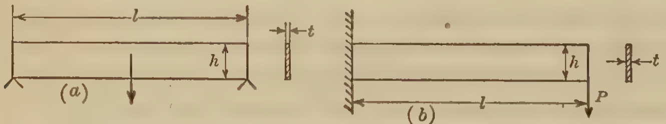

Fig. 7.

$$P_{cr} = \frac{16.9\sqrt{B_2\,C}}{l^2} \quad \text{......................} (21)$$

In the case of a cantilever beam (Fig. 7 (b)):

$$P_{cr} = \frac{4.01\sqrt{B_2\,C}}{l^2} \quad \text{......................} (22)$$

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* *Transactions*, Am. Soc. C. E., Vol. 87 (1924), p. 1247.

† These cases have been discussed by L. Prandtl, in his dissertation, "Kipperscheinungen," München, 1899.

If h is the depth, t, the thickness of the web, and G, the modulus of shear,

$$B_2 = \frac{E\,h\,t^3}{12} \text{ and } C = \frac{1}{3}\,G\,h\,t^3$$

Substituting these in Equations (21) and (22), the critical load for each particular case easily can be calculated.

VI.—BUCKLING OF COMPRESSED PLATES

A.—In compression members of large cross-section, buckling of compressed plates may be of practical importance. Consider, for instance, the cross-sections given in Fig. 10. By diminishing the thickness of the column wall the member will fail under compression by the sidewise buckling of the plates themselves and not by the deflection of the column as a whole. The sides of columns may be considered as long rectangular plates under compression and with various conditions of fixity along the edges. In the simplest case a rectangular plate simply supported is compressed by forces uniformly distributed along the edges, $x = 0$ and $x = a$ (Fig. 8). When the compressive stress exceeds certain limits (critical compressive stress), the flat form of the plate becomes unstable and lateral buckling will occur. By application of the energy method, the equation for the critical stress will be:

$$p_{cr} = \alpha_1\,p_e \quad \text{.........................} (23)$$

in which,

$$p_e = \frac{E\,\pi^2\,t^2}{12\,h^2\,(1 - \sigma^2)} \quad \text{.......................} (24)$$

h = the width of the plate.
σ = Poisson's ratio for the material.
α_1 = a numerical factor depending on the ratio, $\frac{a}{b}$.

It is easy to see that p_e represents the critical compressive stress as obtained by an equation analogous to Equation (1) for a strip having the length, h, and the thickness, t, equal to the thickness of the plate. In Table 4 several values of α_1 for various ratios, $\frac{a}{h}$, are given.

TABLE 4.—VALUES OF α_1 IN EQUATION (23) FOR RECTANGULAR PLATE SIMPLY SUPPORTED ON ALL SIDES.

$\frac{a}{h}$ =	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0
α_1 =	8.41	5.14	4.20	4.00	4.13	4.47	4.20	4.04	4.00
p_{cr}, in pounds per square inch =	22 800	14 000	11 400	10 900	11 200	12 100	11 400	11 000	10 900

It is seen that α_1 is a minimum for a square plate. If a long plate buckles under compression it subdivides in squares or in rectangles, almost square. For each of these rectangles Equation (23) can be applied, the length, a, being the distance between the two consecutive nodal lines parallel to the y-axis.

For a longer plate, say, when $\frac{a}{b} > 3$, a value, $\alpha_1 = 4$, always represents a good approximation.

In Table 4 the values, p_{cr}, are calculated on the assumption that $E = 30 \times 10^6$ lb. per sq. in., $\sigma = 0.3$, and $\frac{h}{t} = 100$. Taking into consideration that the critical stress is proportional to $\frac{t^2}{h^2}$ (Equation (24)), the critical stress for any value of $\frac{h}{t}$, different from 100, is obtained by multiplying the tabular values by $10^4 \frac{t^2}{h^2}$. Take, for instance, a long steel plate having a yield point of 40 000 lb. per sq. in. To determine the value of the ratio, $\frac{h}{t}$, at which the critical stress becomes equal to the yield point, assuming $\alpha_1 = 4$: From Table 4,

$$10\,900 \times 10^4 \times \frac{t^2}{h^2} = 40\,000$$

that is,

$$\frac{h}{t} = \sqrt{\frac{10\,900}{4}} = 51 \pm$$

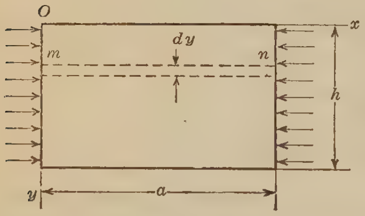

FIG. 8.

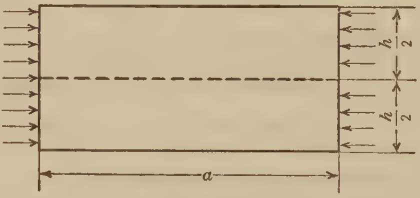

FIG. 9.

For larger values of the ratio, $\frac{h}{t}$, failure will occur as a result of instability and at a compressive stress less than the yield point. This means that in consequence of lateral buckling the complete strength of the material of the plate will not be utilized.

B.—By properly increasing the thickness of the plate, the buckling will not occur below the yield point; but such a uniform increase in thickness is not always the most economical solution. Sometimes, for compressed plates of considerable width, it appears more advantageous to keep the thickness as small as possible and to increase the stability against buckling by introducing longitudinal stiffeners. Such methods of reinforcement are used sometimes in shipbuilding, where the economy in the proper weight of the hull is of great practical importance. In proportioning such reinforcement, it is important to know how the stability of the reinforced plate depends on the flexural rigidity and cross-sectional area of the stiffeners. This problem can be solved by using the energy method.*

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* For detailed investigation of this question, see the writer's paper in *Der Eisenbau*, Vol. 12 (1921), p. 147.

If the plate is reinforced by one longitudinal stiffener at the middle of the width (Fig. 9), the maximum reinforcement occurs when the flexural rigidity of the stiffener is sufficient to prevent it from bending during buckling of the plate. Then each half of the plate will be in the condition of a plate of the width, $\frac{h}{2}$. At smaller values of the flexural rigidity the stiffener bends with the plate during buckling and the value of the critical compressive stress will depend on the ratios:

$$\frac{a}{h}; \; \frac{B}{h\,D}; \; \frac{A}{h\,t} \qquad (25)$$

in which,

$B = E\,I$ = flexural rigidity of the stiffener in bending at right angles to the plane of the plate.

$h\,D = h \dfrac{E\,t^3}{12\,(1-\sigma^2)}$ = the flexural rigidity of the plate.

A = cross-sectional area of the stiffener.

$h\,t$ = cross-sectional area of the plate.

For calculating the critical compressive stress, Equation (23) can be used. The values of the numerical factor, α_1, for various values of the ratios in Equations (25) are given in Table 5.

Numerical Example.—Let a compressed steel plate with simply supported edges have the following dimensions: $a = 48$ in.; $h = 80$ in.; $t = \frac{9}{16}$ in.; $E = 30 \times 10^6$ lb. per sq. in.; and $\sigma = 0.3$. Then, $\frac{a}{h} = 0.6$ and from Table 4,

$$p_{cr} = 14\,000 \times 10^4 \times \frac{9^2}{16^2\,80^2} = 6\,900 \pm \text{ lb. per sq. in.}$$

By using a stiffener of such a flexural rigidity that it does not bend while the plate is buckling, the critical stress becomes:

$$p_{cr} = 11\,200 \times \frac{9^2}{16^2\,40^2} = 22\,100 \pm \text{ lb. per sq. in.}$$

From Table 5 it is seen that $\left(\text{for } \frac{a}{h} = 0.6\right)$ such a condition can be obtained by using a stiffener, for which $\frac{B}{h\,D} = 5$ and $\frac{A}{h\,t} < 0.2$. In calculating the quantity, B, note that the stiffener is riveted to a plate of great width and this results in a considerable increase of rigidity. By taking, for instance, the stiffener in the form of a channel, or a **Z**, riveted to the plate by its flange, the center of gravity of the section consisting of the stiffener and the plate will be very near to the surface of the plate, and the moment of inertia of the cross-section of the stiffener with respect to the axis coinciding with the outer surface of the flange must be taken in calculating B. Using two longitudinal stiffeners, sub-dividing the plate into three equal parts, Equation (23) still applies for calculating the critical compressive stress. The corresponding values of the factor, α_1, are given in Table 6.

TABLE 5.—VALUES OF THE FACTOR, α_1, EQUATION (23), FOR A COMPRESSED RECTANGULAR PLATE REINFORCED BY A LONGITUDINAL STIFFENER.

$\frac{a}{h}$.	$\frac{B}{h\,D} = 5.$			10.			15.			20.			25.		
	$\frac{A}{h\,t} = 0.05.$	0.1.	0.2.	0.05.	0.1.	0.2.	0.05.	0.1.	0.2.	0.05.	0.1.	0.2.	0.05.	0.1.	0.2.
0.6	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
0.8	15.4	14.6	13.0	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8	16.8
1.0	12.0	11.1	9.72	16	16	15.8	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0	16.0
1.2	9.83	9.06	7.88	15.3	14.2	12.4	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5	16.5
1.4	8.62	7.91	6.82	12.9	12.0	10.3	16.1	15.7	13.6	16.1	16.1	16.1	16.1	16.1	16.1
1.6	8.01	7.38	6.32	11.4	10.5	9.05	14.7	13.6	11.8	16.1	16.1	14.4	16.1	16.1	16.1
1.8	7.84	7.19	6.16	10.6	9.70	8.35	13.2	12.2	10.5	15.9	14.7	12.6	16.2	16.2	14.7
2.0	7.96	7.29	6.24	10.2	9.35	8.03	12.4	11.4	9.80	14.6	13.4	11.6	16.0	15.4	13.3
2.2	2.28	7.58	6.50	10.2	9.30	7.99	12.0	11.0	9.45	13.9	12.7	10.9	15.8	14.5	12.4
2.4	8.79	8.06	6.91	10.4	9.49	8.15	11.9	10.9	9.37	13.5	12.4	10.6	15.1	13.8	11.9
2.6	9.27	8.50	7.28	10.8	9.86	8.48	12.1	11.1	9.53	13.5	12.4	10.6	14.8	13.6	11.6
2.8	8.62	7.91	6.31	11.4	10.4	8.94	12.5	11.5	9.85	13.7	12.6	10.8	14.8	13.6	11.6
3.0	8.31	7.62	6.53	12.0	11.1	9.52	13.1	12.0	10.3	14.1	13.0	11.1	15.2	13.9	11.9
3.2	8.01	7.38	6.32	11.4	10.5	9.05	13.9	12.7	10.9	14.8	13.5	11.6	15.6	14.3	12.3
3.6	7.84	7.19	6.16	10.6	9.70	8.35	13.2	12.2	10.5	15.9	14.7	12.6	16.2	15.7	13.5
4.0	7.96	7.29	6.24	10.2	9.35	8.03	12.4	11.4	9.8	14.6	13.4	11.6	16.0	15.4	13.3

C.—When the edges of the plate parallel to the direction of compression, instead of being simply supported, are built in, the stability of the plate is increased. For calculating the critical values of compressive stress, Equation (23) can be used, but the values of the factor, α_1, must be taken from Table 7. It is seen that the minimum value of the factor, α_1, corresponds to the ratio, $\frac{a}{h} = 0.7 \pm$, so that a long compressed plate, when it buckles, subdivides into rectangles approaching these proportions. For a plate of considerable length, say, $\frac{a}{h} > 2$, $\alpha_1 = 7.00$ is always a good approximation.

TABLE 6.—VALUES OF THE FACTOR, α_1, IN EQUATION (23) FOR A RECTANGULAR PLATE REINFORCED BY TWO LONGITUDINAL STIFFENERS.

$\frac{a}{h}$	$\frac{B}{hD} = \frac{10}{3}$		5		$\frac{20}{3}$		10	
	$\frac{A}{ht} = 0.05$	0.1	0.05	0.1	0.05	0.1	0.05	0.1
0.6	26.8	24.1	36.4	33.2	36.4	36.4	36.4	36.4
0.8	16.9	15.0	23.3	20.7	29.4	26.3	37.2	37.1
1.0	12.1	10.7	16.3	14.5	20.5	18.2	28.7	25.6
1.2	9.61	8.51	12.6	11.2	15.5	13.8	21.4	19.0
1.4	8.32	7.36	10.5	9.32	12.7	11.3	17.2	15.2
1.6	7.70	6.81	9.40	8.31	11.1	9.82	14.5	12.8
1.8	7.51	6.64	8.85	7.83	10.2	9.02	12.9	11.4
2.0	7.61	6.73	8.70	7.69	9.78	8.65	11.9	10.6

D.—When one longitudinal side ($y = 0$) of the plate shown in Fig. 8 is built in and another ($y = h$) is free, the critical value of the uniform compressive stress can be calculated again from Equation (23). The values of the numerical factor, α_1, for this case are given in Table 8.

TABLE 7.—VALUES OF THE FACTOR, α_1, IN EQUATION (23) FOR A RECTANGULAR PLATE, THE TWO SIDES OF WHICH, PARALLEL TO COMPRESSION, ARE BUILT IN AND TWO OTHERS ARE SIMPLY SUPPORTED.

$\frac{a}{h} =$	0.4	0.5	0.6	0.7	0.8	0.9	1.0
$\alpha_1 =$	9.44	7.69	7.05	7.00	7.29	7.83	7.69
p_{cr}, in pounds per square inch, for $\frac{h}{t} = 100 =$	25 000	20 800	19 100	19 000	19 800	21 200	20 800

It is seen that the minimum value of the factor, α_1, corresponds to a value of the ratio, $\frac{a}{h}$, about 1.7. A long plate, while buckling, will subdivide in rectangles approaching these proportions. Taking $\alpha_1 = 1.33$ and the yield point as 40 000 lb. per sq. in., for a long steel plate ($E = 30 \times 10^6$ lb. per sq in.; $\sigma = 0.3$), the limiting value of the ratio, $\frac{h}{t}$, at which the critical stress and yield point coincide, will be:

$$p_{cr} = 1.33 \frac{E \pi^2}{12 (1 - \sigma^2)} \frac{t^2}{h^2} = 40\,000$$

from which, $\frac{h}{t} = 30 \pm$.

TABLE 8.—VALUES OF THE FACTOR, α_1, FOR A COMPRESSED RECTANGULAR PLATE, ONE OF THE LONGITUDINAL EDGES OF WHICH IS BUILT IN AND ANOTHER FREE.

$\frac{a}{h} =$	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.2	2.4
$\alpha_1 =$	1.70	1.56	1.47	1.41	1.36	1.34	1.33	1.33	1.34	1.36	1.38	1.45	1.47

For a larger value of the ratio, $\frac{h}{t}$, the buckling of the plate will occur at a stress less than the yield point of the material and for improving the conditions the reinforcement of the free edge of the plate by riveting a stiffener becomes necessary. By using Tables 4, 5, and 8, the necessary thickness of steel sheets for built-up sections of compressed members, such as those shown in Fig. 10, can easily be calculated.

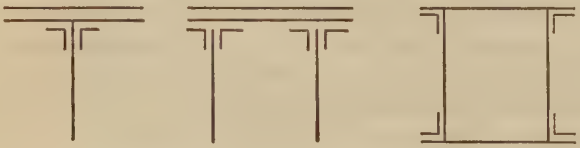

FIG. 10.

VII.—STABILITY OF THE WEB OF A PLATE GIRDER

In the design of plate girders, the thickness of the web is usually calculated for safety in shearing. The calculation does not give sufficient thickness and in order to prevent buckling usually it is necessary to increase the thickness and to use stiffeners, without considering the stability of the stressed web. This results in types of construction having widely varying factors of safety with respect to buckling. For example, plate girders of one large American railroad system, with a span, $l = 100$ ft., and with a depth, $h = 10$ ft., have a web thickness, t, equal to $\frac{3}{8}$ in., while similar girders of another railroad have a web thickness of $\frac{7}{16}$ in. near the supports and only $\frac{3}{8}$ in. at the middle. These girders, both satisfying American specifications, have very different factors of safety with regard to the buckling of the web.

A more satisfactory design can be obtained from a consideration of the stability of the stressed web. Near the supports the shearing force is the most important factor, and stresses due to bending can be neglected. Considering the part of the web between two stiffeners as a rectangular plate with supported edges, subjected only to the action of shear (Fig. 11), the critical value of shearing stress can be found from consideration of the energy, and the equation for calculating this value has the same form as Equation (23). The corresponding values of the numerical factor, α_1, are given in Table 9.

TABLE 9.—VALUES OF THE FACTOR, α_1, IN EQUATION (23) FOR A RECTANGULAR PLATE SUPPORTED AT THE EDGES AND SUBMITTED TO SHEAR.

$\frac{h}{a} =$	1	1.2	1.4	1.5	1.6	1.8	2.0	2.5	3.0
$\alpha_1 =$	9.42	8.0	7.3	7.1	7.0	6.8	6.6	6.3	6.1

Applying these results to steel plates ($E = 30 \times 10^6$ lb. per sq. in., $\sigma = 0.3$), and assuming the distance between the stiffeners equal to 5 ft., the critical values of shearing stress for different depths, h, and different thicknesses, t, are as given in Table 10.

TABLE 10.—CRITICAL SHEARING STRESSES, IN POUNDS PER SQUARE INCH.

Values of h, in feet.	VALUES OF t, IN INCHES.			
	⅜	7/16	½	9/16
5	9 980	13 600	17 700	22 400
7	7 730	10 500	13 700	17 400
10	6 990	9 510	10 400	15 700

Assuming the factor of safety as 2 and the average shearing stress at the support as 5 000 lb. per sq. in., Table 10 shows that for both the plate girders mentioned, a thickness of ½ in. will insure the stability of the web near the supports.*

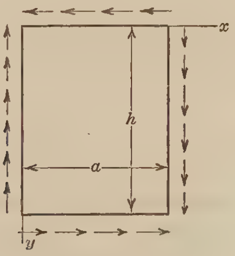

FIG. 11.

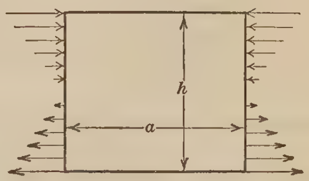

FIG. 12.

It was assumed that the buckling of the web was not accompanied by bending of the stiffeners. This assumption will be true only when the stiffeners have a sufficient flexural rigidity. The necessary magnitude of the moment of inertia of the cross-section of the stiffeners can be found from the potential energy of bending of these stiffeners, giving the data of Table 11.*

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* Under actual conditions the factor of safety will be greater than 2, because the edges of the plate are assumed as simply supported and the stiffening effect of fixed edges is neglected.

The quantity, δ, of Table 11 denotes the ratio, $E\,I : \frac{E\,a\,t^3}{12\,(1 - \sigma^2)}$, in which, $E\,I$ is the necessary flexural rigidity of the stiffeners, and $\frac{E\,a\,t^3}{12\,(1 - \sigma^2)}$ is the flexural rigidity of the part of the web between two consecutive stiffeners.

TABLE 11.

$\frac{a}{h} =$	1.0	0.750	0.625	0.500
$\delta =$	3.3	11.6	25.2	60

Taking, for instance, $a = 5$ ft., $h = 10$ ft., and $t = \frac{1}{2}$ in.:

$$I = 60\,\frac{60\left(\frac{1}{2}\right)^3}{12\,(0.91)} = 41 \pm \text{in.}^4$$

This is the necessary value for the moment of inertia of the cross-section of the stiffeners.

At the middle of the span the shearing stresses can be neglected in comparison with normal stresses. Then, the part of the web between two stiffeners will be in the condition represented in Fig. 12. Applying the same general energy method, Equation (23) will be found for calculating critical values of bending stresses. The corresponding values of the factor, α_1, are given in Table 12.

TABLE 12.—VALUES OF THE FACTOR, α_1, FOR A SIMPLY SUPPORTED RECTANGULAR PLATE STRESSED AS SHOWN IN FIG. 12.

$\frac{a}{h} =$	0.40	0.50	0.60	0.667	0.75	0.80	1.0	1.5
$\alpha_1 =$	29.1	25.6	24.1	23.9	24.1	24.4	25.6	24.1

The minimum value of α_1 will be for the ratio, $\frac{a}{h}$, equal to 0.67. Evidently, however, the placing of stiffeners closer together at the middle of the span will not affect materially the stability of the web at this point.

Applying Table 12 in the case of the plate girder having $a = 5$ ft., $h = 10$ ft., and $t = \frac{3}{8}$ in. at the middle,

$$p_{cr} = 25.6\,\frac{E\left(\frac{3}{8}\right)^2}{(12)\,(120)^2\,(0.91)} = 6\,800 \pm \text{lb. per sq. in.}$$

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* See the writer's paper in *Der Eisenbau*, Vol. 12 (1921), p. 147.

This stress is smaller than that generally considered safe for plate girders. It was assumed that the edges were simply supported; in practice they are always rigidly connected with the flanges. Therefore, the actual critical stresses will be somewhat greater than the theoretical. Nevertheless, probably some buckling of the web occurs under ordinary loading conditions. This buckling does not represent an immediate danger to the girder, but it causes an overstressing of the girder flanges and of the rivets, which is undesirable.

APPENDIX

STABILITY OF THE COMPRESSED TOP CHORD OF A HALF-THROUGH TRUSS SPAN

Considering the compressed top chord of a half-through bridge as a strut of uniform cross-section having hinged ends, and assuming that the compressive forces are continuously distributed, as shown by the shaded area in Fig. 6(*b*), the critical value of the compressive force at the middle can be calculated by using the energy method. In this manner an equation analogous to Equation (9) can be established. Equations (6) and (7) for the potential energy of bending and the potential energy of deformation of the elastic medium can be used also in this case. In calculating the work done by the compressive forces during lateral buckling (Fig. 13) assume that Support B is on rollers so that during buckling it moves toward Support A. Due to displacement of one element, $d\,x$, of the strut (Fig. 13) the loads acting on the right-hand portion of the strut are displaced toward Support A by an amount equal to $d\,s - d\,x = \frac{1}{2}\left(\frac{d\,y}{d\,x}\right)^2 d\,x$, and the work produced during this displacement will be,

$$-\frac{1}{2}\left(\frac{d\,y}{d\,x}\right)^2 d\,x \int_x^l \frac{q\,l}{2\,h}\left(1-\frac{2\,x}{l}\right) d\,x = \frac{q}{4\,h}\left(\frac{d\,y}{d\,x}\right)^2 (l-x)\,x\,d\,x$$

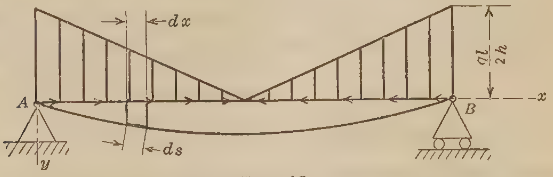

FIG. 13.

The complete work given by the compressive forces during bending will be:

$$T = \frac{q}{4\,h}\int_0^l \left(\frac{d\,y}{d\,x}\right)^2 (l-x)\,x\,d\,x \ldots\ldots\ldots\ldots\ldots\ldots (A)$$

so that the equation for determining the critical value of the compressive force, $\frac{q\,l^2}{8\,h}$, will have the form:

$$\frac{q}{4\,h}\int_0^l \left(\frac{d\,y}{d\,x}\right)^2 (l-x)\,x\,d\,x = \frac{1}{2}\,E\,I\int_0^l \left(\frac{d^2\,y}{d\,x^2}\right)^2 d\,x + \frac{1}{2}\,K\int_0^l y^2\,d\,x \ldots\ldots\ldots\ldots\ldots\ldots (B)$$

The deflection curve in this case is not a simple sine curve, but it can always be assumed to be represented in the form of the series:

$$y = a_1 \sin \frac{\pi x}{l} + a_2 \sin \frac{2 \pi x}{l} + a_3 \sin \frac{3 \pi x}{l} + \dots\dots\dots\dots (C)$$

each term of which satisfies the conditions at the ends of the strut. Substituting this series in Equation (B) the right-hand member becomes:

$$\frac{1}{2} E I \int_0^l \left(\frac{d^2 y}{d x^2}\right)^2 d x + \frac{1}{2} K \int_0^l y^2 d x$$

$$= \frac{E I \pi^4}{4 l^3} \sum_{n = 1, 2, 3 \dots\dots} n^4 a_n{}^2 + \frac{K l}{4} \sum_{n = 1, 2, 3 \dots\dots} a_n{}^2 \dots\dots\dots\dots\dots (D)$$

In calculating the left-hand member of Equation (B), the following formulas will be used:

$$\int_0^l x \cos^2 \frac{m \pi x}{l} d x = \frac{l^2}{4}$$

$$\int_0^l x^2 \cos^2 \frac{m \pi x}{l} d x = \frac{l^3}{6} + \frac{l^3}{4 \pi^2 m^2}$$

in which, m is an integer.

$$\int_0^l x^2 \cos \frac{n \pi x}{l} \cos \frac{m \pi x}{l} d x = \frac{2 l^3}{\pi^2} \frac{(m^2 + n^2)}{(m^2 - n^2)} 2 (-1)^{m+n}$$

in which, m and n are both integers.

$$\int_0^l x \cos \frac{n \pi x}{l} \cos \frac{m \pi x}{l} d x = 0$$

when $m + n$ is an even number; and,

$$\int_0^l x \cos \frac{n \pi x}{l} \cos \frac{m \pi x}{l} d x = - \frac{2 l^2}{\pi^2} \frac{m^2 + n^2}{(m^2 - n^2)^2}$$

when $m + n$ is an odd number.

Then,

$$\frac{q}{4 h} \int_0^l \left(\frac{d y}{d x}\right)^2 (l - x) x d x = \frac{q l}{4 h} \left\{ \sum_{n = 1, 2, 3 \dots\dots} a_n{}^2 \left(\frac{n^2 \pi^2}{12} - \frac{1}{4}\right) \right.$$

$$\left. - 4 \sum \sum_{(m + n) \text{ even} \dots\dots} a_n a_m \frac{n m (m^2 + n^2)}{(m^2 - n^2)^2} \right\} \dots\dots\dots\dots\dots\dots (E)$$

Substituting values from Equations (D) and (E) in Equation (B),

$$\frac{q l^2}{8 h} = \frac{\dfrac{E I \pi^4}{8 l^2} \displaystyle\sum_{n = 1, 2, 3 \dots\dots} n^4 a_n{}^2 + \dfrac{K l^2}{8} \displaystyle\sum_{n = 1, 2, 3 \dots\dots} a_n{}^2}{\displaystyle\sum_{n = 1, 2, 3 \dots\dots} a_n{}^2 \left(\frac{n^2 \pi^2}{12} - \frac{1}{n}\right) - 4 \sum \sum_{(m + n) \text{ even} \dots\dots} a_n a_m \frac{n m (m^2 + n^2)}{(m^2 - n^2)^2}} \dots\dots (F)$$

In order to get the critical value of the compressive force, there only remains the coefficients, a_1, a_2, a_3 . . ., to be determined in such a manner that Equation (F) will be a minimum. By making the derivatives of this expression with respect to a_1, a_2, a_3 . . ., equal to zero, a system of equations will be found having the following form:

$$a_n \left[\pi^2 n^4 + \alpha \pi^2 - \gamma \left(\frac{2}{3} \pi^2 n^2 - 2\right)\right] + 16 \gamma \sum_{(m+n)\text{ even}}\sum a_m \frac{n\, m\, (m^2 + n^2)}{(m^2 - n^2)^2} = 0 \quad \ldots\ldots\ldots\ldots\ldots (G)$$

in which, for simplification, the following notations are used:

$$\alpha = \frac{K\, l^4}{E\, I\, \pi^4}\,;\ \gamma = \text{ratio of } \frac{q\, l^2}{8\, h} : \frac{E\, I\, \pi^2}{l^2} \quad \ldots\ldots\ldots\ldots\ldots (H)$$

By taking for n the values, 1, 3, 5, . . ., the following equations result:

$$\left.\begin{aligned}
&a_1 \left[\pi^2 + \alpha \pi^2 - \gamma \left(\frac{3}{2} \pi^2 - 2\right)\right] \\
&\quad + \gamma \left(\frac{15}{2} a_3 + \frac{65}{18} a_5 + \frac{175}{72} a_7 + \ldots\right) = 0 \\
&a_1 \frac{15}{2} \gamma + a_3 \left[\pi^2 3^4 + \alpha \pi^2 - \gamma \left(\frac{2}{3} 9 \pi^2 - 2\right)\right] \\
&\quad + \gamma \left(\frac{255}{8} a_5 + \frac{609}{50} a_7 + \ldots\right) = 0 \\
&a_1 \frac{65}{18} \gamma + a_3 \frac{255}{8} \gamma + a_5 \left[\pi^2 5^4 + \alpha \cdot \pi^2 - \gamma \left(\frac{2}{3} 5^2 \pi^2 - 2\right)\right] \\
&\quad + \gamma \left(\frac{1295}{18} a_7 + \ldots\right) = U \\
&a_1 \frac{175}{72} \gamma + a_3 \frac{609}{50} \gamma + a_5 \frac{1295}{18} \gamma \\
&\quad + a_7 \left[\pi^2 7^4 + \alpha \pi^2 - \gamma \left(\frac{2}{3} 7^2 \pi^2 - 2\right)\right] + \ldots = 0
\end{aligned}\right\} \ldots\ldots\ldots (I)$$

Only the coefficients, a_n, with odd subscripts enter into Equations (*I*) and, as seen from Equation (*C*), only deflection curves symmetrical with respect to the middle of the span correspond to these equations.

By taking for n the consecutive even values, 2, 4, 6, . . ., the following equations are found:

$$\left.\begin{aligned}
&a_2 \left[\pi^2 2^4 + \alpha \pi^2 - \gamma \left(\frac{2}{3} \pi^2 2^2 - 2\right)\right] \\
&\quad + \gamma \left(\frac{160}{9} a_4 + \frac{15}{2} a_6 + \ldots\right) = 0 \\
&a_2 \frac{160}{9} \gamma + a_4 \left[\pi^2 4^4 + \alpha \pi^2 - \gamma \left(\frac{2}{3} \pi^2 4^2 - 2\right)\right] \\
&\quad + \gamma \left(\frac{1248}{25} a_6 + \ldots\right) = 0 \\
&a_2 \frac{15}{2} \gamma + a_4 \frac{1248}{25} \gamma + a_6 \left[\pi^2 6^4 + \alpha \pi^2 \right. \\
&\quad \left. \gamma \left(\frac{2}{3} \pi^2 6^2 - 2\right)\right] + \ldots = 0
\end{aligned}\right\} \ldots\ldots\ldots (J)$$

Equations (J) contain only the coefficients, a_n, with even subscripts; hence, the corresponding deflection curves have an inflection point at the middle.

It is clear that in order for buckling to occur, at least one or more of the values of a_n given by Equations (I) or (J) must be different from zero. This condition will require that the determinant of Equations (I) or (J) become equal to zero. The critical values of the compressive force may now be calculated by the following method of successive approximations.

Begin with the case of a very small K; then, as found from the discussion of the simple case in Part III, the deflection curve consists of one wave only and the system of Equations (I) must be used for the calculation of the critical compressive forces. Take, for instance, $K = 0$. Making all the coefficients, a_n, except a_1, equal to zero and retaining only the first equation of the system, (I),

$$\pi^2 - \gamma\left(\frac{2}{3}\pi^2 - 2\right) = 0, \text{ or } \gamma = \frac{\pi^2}{\frac{2}{3}\pi^2 - 2} = 2.15$$

This first approximation, equivalent to the assumption that the deflection curve is a sine wave, is somewhat greater than the more accurate value, $\gamma = 2.06$ given in Table 2. The error of the first approximation is about 4½ per cent.

In order to get a better approximation take two consecutive coefficients, a_1 and a_3, and put the remaining coefficients equal to zero, in the system of Equations (I). Then, the first two equations of this system become:

$$a_1\left[\pi^2 - \gamma\left(\frac{2}{3}\pi^2 - 2\right)\right] + a_3\frac{15}{2}\gamma = 0$$

$$a_1\frac{15}{2}\gamma + a_3\left[\pi^2 3^4 - \gamma\left(\frac{2}{3}\pi^2 9 - 2\right)\right] = 0$$

These equations will yield for a_1 and a_3 a solution different from zero, and buckling will occur only when the following condition is fulfilled:

$$\left[\pi^2 - \gamma\left(\frac{2}{3}\pi^2 - 2\right)\right]\left[\pi^2 3^4 - \gamma\left(\frac{2}{3}\pi^2 9 - 2\right)\right] - \left(\frac{15}{2}\gamma\right)^2 = 0$$

Solving this quadratic equation, $\gamma = 2.06$. This second approximation is the value given in Table 2. Calculation of the third approximation shows that in this case the second approximation gives three correct figures for the factor, γ.

In the same manner, calculations were made for,

$$\frac{K\,l^4}{16\,E\,I} = 5;\ \frac{K\,l^4}{16\,E\,I} = 10;\ \frac{K\,l^4}{16\,E\,I} = 15$$

and the corresponding values of γ are given in Table 2. For larger values of $\frac{K\,l^4}{16\,E\,I}$, Equations (J) must be used, because in such cases the deflection curve will have two waves with an inflection point at the middle. With further increase of K curves will be obtained with three, four, and more waves. It is clear that for an odd number of waves, Equations (I), and for an even number, Equations (J), must be used in calculating the critical loads.

DISCUSSION

H. M. WESTERGAARD,[*] M. AM. SOC. C. E.—The author has written a number of scientific papers on the subject of elastic stability, and has given a notable development to the technique of solving problems in this field. He has used extensively a general principle of equilibrium which is based on the variations of the potential energy of the deformed structure and the loads. The systematic use of this principle has been fruitful.[†] The particular problems treated in this paper are unquestionably important.

The principle of variation of the potential energy lends itself to mathematically exact solutions of the simpler problems and to approximate solutions of the more complex ones. Generally speaking, when approximations are admitted, the problems shift from mathematics to physics. In this connection a remark by the late C. Christiansen, Professor of Physics in Copenhagen, deserves to be quoted: "There is the difference", he said, "between mathematics and physics, that in mathematics everything is right and in physics everything is wrong." A method which belongs partly in mathematics and partly where "everything is wrong" requires mathematical technique of course, but it also requires sound judgment, as well as common sense on the part of the engineer or the physicist. These requirements are not easy to meet, but in the speaker's opinion, this paper meets both of them. It may be added that a sound judgment is expected also of those who are to interpret the results for the purpose of structural design.

The paper contains definite answers to definite questions which may be asked by the structural engineer. Answers are stated in terms of tabulated numerical values which are to be substituted in simple algebraic expressions.

After receiving these definite answers, it is reasonable to ask where one stands. This issue may be discussed by referring to the most familiar problem of elastic stability. Fig. 14 (*a*) shows diagrams of load and deflections for a slender column. The combination, *O A B,* represents what would happen if the column were perfectly straight, homogeneous, and centrally loaded. In a test, in which these conditions of the ideal column cannot be entirely satisfied, one would find, perhaps, the curve, *O C D.* These diagrams are characteristic of the buckling of slender structures in general. The ordinate, *O A,* represents the computed critical load. Under practical conditions one must expect a curve like *O C D.* It may be stated, however, that knowledge of the position of the horizontal line, *A B,* helps in forming a judgment as to the position of the curve, *C D.*

If the column is not particularly slender, but of medium length, one may obtain the curves shown in Fig. 14 (*b*): *O A B* for the ideal column, the part, *A B,* being curved because some of the stresses exceed the proportional limit; and *O C D* in a test, or in practice. Grant that the critical load, *O A,* has been computed, in spite of the difficulties which accompany a reduced modulus of elasticity. It is the ordinate of the apex, *C*, which is of practical interest. This true maximum load may be considerably less than the load, *O A*, obtained under idealized conditions. Th. von Kármán studied these relations for steel columns in an investigation published in 1910.*

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[†] For extensive bibliographical information, see the chapter by Professor Timoshenko under the heading, "Stabilitätsprobleme der Elastizität," in Handbuch der physikalischen und technischen Mechanik, v. 4, Lieferung 1, 1929, pp. 81–145.

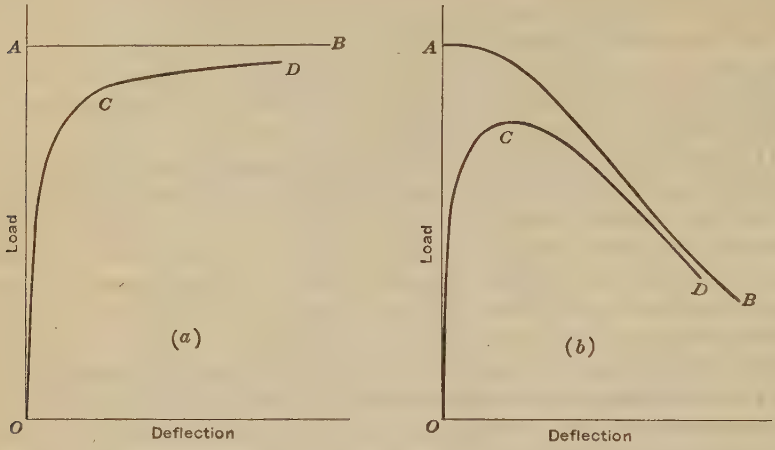

FIG. 14.

The diagrams in Fig. 14 (*b*) are characteristic of the buckling of the less slender structures in general. Professor Timoshenko refers occasionally to these less slender structures. It is true that an estimate of the position of Point *A* in Fig. 14(*b*) may help in forming a judgment as to the position of Point *C*. The problem involved here still represents a field for future research, both experimental and analytical.

The problems of elastic stability of slender structures, in which stresses exceeding the proportional limit need not be considered, are, of course, important for their own sake. It is a peculiar circumstance, however, that a study of the critical loads may lead to the solution of problems which are apparently of a totally different nature. A case, chosen on account of its simplicity, will serve as an illustration.

Fig. 15 (*a*) shows a straight stiff vertical member, with a hinged support at the bottom, supported by a horizontal spring with a modulus, *S*, at the top. When the load, *P*, reaches a critical value, *Q*, the structure may "buckle", as shown in Fig. 15 (*b*). The critical load may be determined either by direct consideration of the forces involved or by the principle of the potential energy. By the latter method one finds that the deflection in the direction of Q is equal to $\frac{u^2}{2\,l}$; then, the potential energy may be expressed as $\frac{1}{2}\,S\,u^2 - Q\,\frac{u^2}{2\,l}$. This potential energy remains zero, for any value of u, when $Q = S\,l$, which therefore, is the critical load. The same deflection, u, may be produced by the horizontal load, W, in Fig. 15 (c). If, as in Fig. 15(d), the load, P (less than Q), is introduced in addition to W, the deflection (as one finds from the condition that the potential energy is a minimum), becomes $u' = u \dfrac{Q}{Q - P}$, instead of u.

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* Th. von Kármán, "Untersuchungen über Knickfestigkeit," *Forschungsarbeiten* auf dem Gebiete des Ingenieurwesens, Heft 81, 1910; see, also, M. Ros, *Proceedings*, Second International Cong. for Applied Mechanics, Zürich, 1926 (pub. 1927), p. 368; and a paper by William R. Osgood, Assoc. M. Am. Soc. C. E., and the speaker, *Transactions*, Am. Soc. Mech. Engrs., v. 50, 1928, No. 17, p. 65.

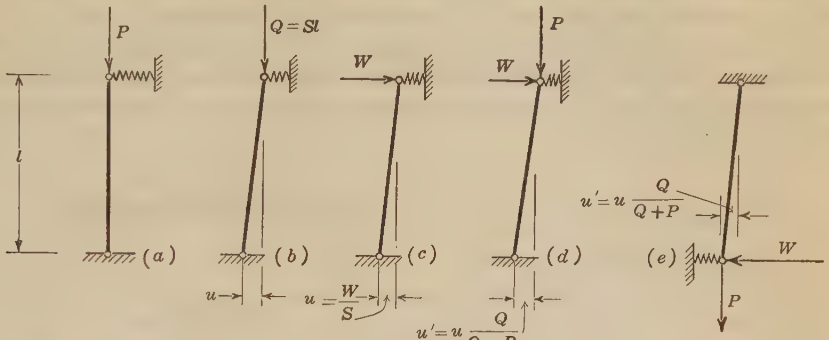

FIG. 15.

Now, turn this structure upside down, and load it as shown in Fig. 15(e). The same formula may be used, only with the sign of P reversed. The weight, P, in this case, steadies the structure, and this steadying effect is evaluated in terms of the load which produces buckling.

A steadying effect of this kind plays an important part in the mechanics of suspension bridges. This effect expresses itself in terms of the difference between what have been called the "elastic theory" and the "deflection theory". Melan* originated the deflection theory as contrasted with the ordinary theory; the latter becomes incorrect and wasteful of material when applied to large suspension bridges. Leon S. Moisseiff,† M. Am. Soc. C. E., in discussing the suspension bridge between Philadelphia, Pa., and Camden, N. J., gave emphasis to the deflection theory, and showed its economic importance. As a matter of fact, the problem involved here may be treated from the point of view of elastic stability or buckling.

Consider the case in Fig. 16 (a). Turn the bridge upside down, as indicated in Fig. 16 (b). At a critical value of the load, P, the structure buckles by the bending of the stiffening beam. One must imagine that the sides of the polygon, which did represent the cable, remain straight, but that the polygon changes shape. By introducing this critical value in a formula similar to that applying to Fig. 15 (e), one may express an essential feature relating to the steadying effect which, in the case of the suspension bridge, is due to the dead load.

The problem referring to the suspension bridge is not quite as simple as that in Fig. 15 (e). The complication expresses itself through the fact that the structure in Fig. 16 (b) may be imagined to buckle in more than one way.

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* "Eiserne Bogenbrücken und Hängebrücken," 1888.

† "The Towers, Cables, and Stiffening Trusses of the Bridge Over the Delaware River between Philadelphia and Camden," *Journal*, Franklin Inst., October, 1925.

The principle involved may be studied by examining a structure which is almost as simple as that in Fig. 15. Fig. 16 (*a*) shows this structure. The two stiff vertical members are connected by a hinge, and are supported by two horizontal springs of the same modulus, S. The lower member has a hinged support at the bottom. The structure supports a load, P, at the top. It may be imagined to buckle in the following two ways: Either as shown in Fig. 17 (*b*), under the critical load $Q_1 = 0.382\, S\, l$; or, as shown in Fig. 17 (*c*), under the larger critical load, $Q_2 = 2.618\, S\, l$. These critical values of P, and the corresponding relative deflections shown in the diagram, may be obtained either by a direct study of the forces involved, or by use of the principle of the potential energy.

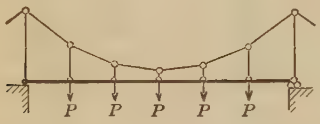

FIG. 16(*a*).

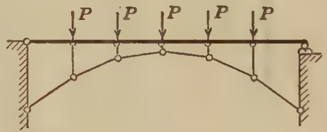

FIG. 16(*b*).

If the problem were merely that of simple buckling, only the smaller critical load, Q_1, would be of interest. Here, however, both critical loads should be considered. The deflections in Fig. 17 (*b*) and Fig. 17 (*c*), may be produced by applying instead the horizontal forces shown in Fig. 17 (*d*) and Fig. 17 (*e*). Fig. 17 (*f*) shows a combination of the loads in Fig. 17 (*a*), Fig. 17 (*d*), and Fig. 17 (*e*). Any combination of two forces, F_1 and F_2, may be expressed by the formulas in Fig. 17 (*f*), in terms of the values, W_1 and W_2, and the corresponding values of the deflections, u_1, v_1, u_2, v_2, in Fig. 17 (*d*) and Fig. 17 (*e*), may be computed. These values, substituted in the formulas in

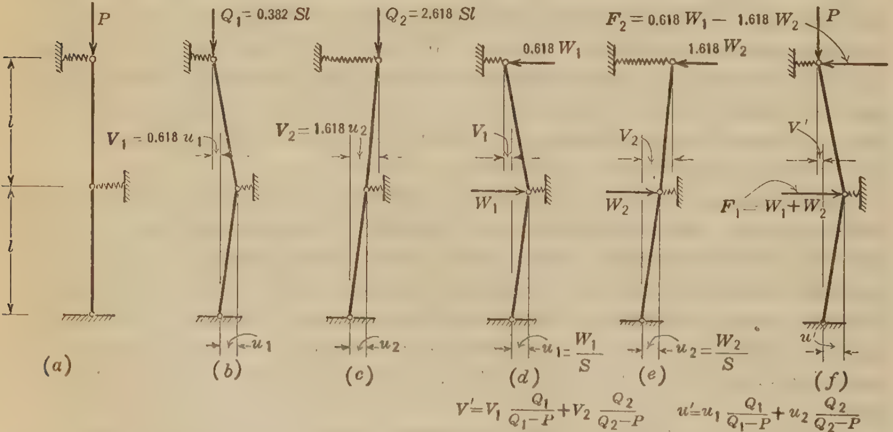
P
l
l
$Q_1 = 0.382\ Sl$
$V_1 = 0.618\ u_1$
u_1
$Q_2 = 2.618\ Sl$
$V_2 = 1.618\ u_2$
u_2
$0.618\ W_1$
V_1
W_1
$u_1 = \frac{W_1}{S}$
$F_2 = 0.618\ W_1 - 1.618\ W_2$
$1.618\ W_2$
V_2
W_2
$u_2 = \frac{W_2}{S}$
P
V'
$F_1 = W_1 + W_2$
u'
(*a*)
(*b*)
(*c*)
(*d*)
(*e*)
(*f*)
$V' = V_1 \frac{Q_1}{Q_1 - P} + V_2 \frac{Q_2}{Q_2 - P}$
$u' = u_1 \frac{Q_1}{Q_1 - P} + u_2 \frac{Q_2}{Q_2 - P}$

FIG. 17.

Fig. 17 (*f*), lead to the resultant deflections, u' and v'. In these formulas the two critical values, Q_1 and Q_2, appear. With three links in the structure, and three springs, instead of two links and two springs, there would be three critical loads, and three terms in each of the expressions in Fig. 17 (*f*). With the structure turned upside down and the weight, P, hanging at the bottom, P again becomes a steadying influence, and the same formulas may be used, only with the sign of P reversed.

This example, of course, merely indicates that formulas of this kind apply to the suspension bridge. In a paper on buckling,* however, the speaker has given a general theory, in the light of which this application to the suspension bridge becomes obvious. In the terminology of this paper the suspension bridge is in "heterostatic action". The general formula (Equation (75)†), for "effects" in heterostatic action, such as deflections, moments, or stresses, gives expressions for the deflections of the suspension bridge that are essentially of the same form as those which Professor Timoshenko derived by a direct method.

Professor Timoshenko's achievements relating to the subject of elastic stability are interesting and important, and they point the way toward future research.

William Hovgaard,‡ Esq. (by letter).—The theory of stability of elastic systems has been developed greatly since the beginning of the Twentieth Century, especially in its application to practical engineering, and in this development Professor Timoshenko has taken a leading part. In this paper he has condensed the solution of a number of problems that are of practical interest to designers, not only among structural engineers, but also among naval architects and others. The value of the paper is enhanced by numerous tables which facilitate the application of the formulas. Throughout all the problems the author uses the energy equation to determine the point of transition between the stable and the unstable state of equilibrium, and it is in particular on the form and derivation of this equation that the writer wishes to comment.

In Part III, the fundamental equation of energy, Equation (9), is given the form:

$$T = V_1 + V_2 \qquad (26)$$

which expresses the statement that the external work done by the compressive forces, $T = P\,\gamma$, shall be equal to the sum of V_1, the elastic work done in bending the strut, and V_2 that done in deformation of the elastic medium. The factor, γ, is the deflection of the point, A, toward the point, B (Fig. 4), caused by bending. Since V_2 is precisely of the same nature as V_1 and since it is here desired to discuss the principle and not any specific case, V_2 is omitted for the sake of simplicity and the energy equation is obtained for a simple column hinged at both ends and without lateral supports. Denoting the elastic work in bending by V instead of V_1, Equation (26) becomes:

$$T = V \qquad (27)$$

Using Equation (27), the critical load according to Equation (11), with the omission of the last term in the brackets, becomes the same as Equation (1).

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* "Buckling of Elastic Structures," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 576, especially Part IV, pp. 604–637.

† *Loc. cit.*, p. 619.

‡ Prof. of Naval Design and Constr., Mass. Inst. Tech., Cambridge, Mass.

Another formula which has been used in connection with the problem of the buckling of plates, is obtained by assuming that the work done by the compressive forces is zero, but that the plate buckles sidewise by a small amount. Thus, there will be introduced an additional potential energy of bending, V, and at the same time the potential energy of compression decreases by an amount, V_1. Hence, if the potential energy of the system is to remain constant, the equation of energy becomes:

$$V - V_1 = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Since both Equations (27) and (28) give the correct result they must be equivalent to each other; but this does not seem immediately obvious. Imagine first that a simple compression displacement, γ_1, takes place in a pin-ended column without any bending, the external work being T, while, at the same time, an internal work, V_c, is done by the elastic compression of the column. Since the total axial stress force in the column is equal to P, $V_c = P\,\gamma_1 = T$. Assume that the column is deflected a very short distance sidewise in bending, so that its axis is lengthened by an increment, γ_2, and a certain amount of energy, V, is absorbed in bending, while, at the same time, an amount of energy, V_1, is released due to the elongation. Equating the external and the internal work:

$$T = V_c + V - V_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

Since V_c is equal to T, Equation (29) may be written, $V - V_1 = 0$, which is the same as Equation (28).

It is thus shown that Equation (29) is equivalent to Equation (28); but it is not quite obvious that it is also always equivalent to Equation (27). Evidently, this will be true only if $V_c = V_1$; that is, if γ_1, the deflection of P, is always equal to the elongation, γ_2, because, in that case, Equation (29) reduces to, $T = V$. In other words, after the column has been compressed a certain distance, γ_1, it shall bend outward so much that exactly the same amount of compression is again taken out of it. This, then, must be assumed to be the case, but it does not seem clear that it is always and necessarily so.

For this reason Equation (28) seems preferable to the writer both in form and mode of derivation, and he proposes the following proof of it, based on the principle of virtual velocities.

Suppose that a column is simply compressed by terminal axial forces, P, acting at each end, and that equilibrium is established. In order to test whether the system is actually in equilibrium, assume that one end of the column is fixed and that P at the other end is moved through a distance, γ_1, so that the column is still further compressed. Here, γ_1 is a small quantity of the first order. Since P remains unchanged, the external work is $T = P\,\gamma_1$. If there is equilibrium, this work must be equal to the internal elastic work, V_c, due to compression. Therefore,

$$T = V_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

In order to determine whether the equilibrium is stable, it is necessary to consider quantities of the second order. The column is still in the compressed state, but both ends are now regarded as fixed and the column is given a sidewise virtual deflection. This deflection is a small quantity of the first order, but the elongation and the bending strains produced thereby are of the second order. A certain amount of energy, V, is absorbed in bending and an amount, V_1, is released due to the elongation caused by bending, but both these quantities are of the second order. Since no external work is done in this case, the condition necessary for stable equilibrium is that $V > V_1$ and if the column is in the state of transition between stable and unstable equilibrium, $V = V_1$, as given in Equation (28). This reasoning is consistent with Equation (29) if in that equation T is made equal to V_c and these quantities are regarded as being of the first order, while V and V_1 are of the second order. The stability of rectangular plates under edge thrust or edge shear is a problem of considerable interest to naval architects, who have not yet made full use of the results achieved in this field of the theory of elasticity. The application of the formulas to ship-plating is complicated, however, by the indefinite mode in which the plates are supported by frames and longitudinals. Furthermore, the presence of riveted seams or butts and the transverse load due to the external water pressures, complicate the problem.

There are at least three cases in which buckling should be considered in the strength of ships. First, when the outer shell or the deck plating is compressed under longitudinal bending of the entire hull; second, when the outer shell plating on the sides is subject to shearing due to forces acting on the entire hull; and, third, when parts of the structure are exposed to excessive local forces, as may occur, for instance, in docking or grounding.

The two first cases are of the greatest importance in lightly built, high-speed vessels, such as fast cruisers and torpedo-boats. In such vessels, the scantlings of the outside plating and of the strength deck are determined primarily with regard to the stresses in longitudinal bending. Generally, a certain maximum calculated stress is allowed or aimed at in the design, but in considering the corresponding factor of safety the strength in buckling or wrinkling of the plating is not generally taken into account. Nor is the choice of spacing of the frames or longitudinals ordinarily influenced by considerations of buckling.

Example 1.—Consider the bottom plating of a torpedo-boat in which the transverse frame spacing, a, is 21 in. and the thickness of the plating, t, is 0.25 in. The spacing, h, of the longitudinal girders is 78 in., and it is desired to examine the effect of buckling pressure when the bottom plating is under longitudinal compression, as in the hogging condition. In a panel enclosed between two adjacent frames and two adjacent longitudinals all the edges are regarded as simply supported. Then, $\frac{h}{a} = \frac{78}{21} = 3.71$; $E = 30 \times 10^6$ lb. per sq. in.; $\sigma = 0.3$; and the flexural rigidity is:

$$D = \frac{E\,t^3}{12\,(1 - \sigma^2)} = \frac{30 \times 10^6\,(0.25)^3}{12 \times 0.91} = 42\,900$$

The critical stress is obtained by means of Equation (31):*

$$p_{cr} = \left(\frac{h}{a} + \frac{a}{h}\right)^2 \frac{\pi^2 D}{t\,h^2} = \alpha_1 \frac{\pi^2 D}{t\,h^2} \dots\dots\dots\dots\dots (31)$$

Making the proper substitutions, the value is, approximately,

$$p_{cr} = 15.8 \times \frac{9.87 \times 42\,900}{0.25 \times 6\,084} = 4\,440 \text{ lb. per sq. in.} = 2 \text{ tons per sq. in.}$$

In vessels of this class it is usual to allow a permissible designed stress in compression of about 6 tons per sq. in. in the extreme fibers, corresponding to perhaps 4 to 5 tons per sq. in. in the strakes of plating here under consideration. Hence, it appears that buckling would be very likely to happen, but, although there are cases on record where buckling resulting in permanent strains has occurred, on the whole, it must be regarded as an exception. Another consideration is that elastic buckling may take place quite commonly in vessels of this type without being noticed because the deformation is arrested by adjacent longitudinal members before over-strain occurs, and when the stresses cease, the plating goes back to its original position. The effect in this case is that certain plates, which form part of the flanges of the ship girder, shirk their duty in compression and greater longitudinal stresses are thrown on the remaining strakes. Such incipient buckling has been previously described by the writer.†

Basing conclusions on various tests and on the practice common in bridge construction, it was estimated that the limiting spacing of longitudinal stiffeners for which the plating may be assumed to be fully effective in compression, should be equal to about eighty times the thickness. If this spacing were exceeded it was recommended to reckon as effective only a band of plating directly attached to the longitudinal stiffeners of a width equal to fifty times the thickness, that is, a strip of width, 25 t, on each side. In the torpedo-boat discussed in Example 1, the ratio, h to t, is more than 300, but in battleships it may be as small as 80 or less, especially in the strength deck.

It must be borne in mind also that in Equation (31), all the edges are assumed to be simply supported, that is, $w = 0$ and $\frac{\partial^2 w}{\partial x^2} = 0$; but whether the last condition is fulfilled seems doubtful. Probably there is considerable clamping effect along all four edges and in such cases buckling will not occur until a greater stress has been reached.

As regards buckling or wrinkling by shearing the writer has heard of no case in which this has occurred due to the general shearing of the entire hull of a ship. If it did occur, it would probably take the form of wrinkling between transverse frames, but although permanent damage of this kind may not have been observed it is still possible that very light elastic wrinkling may have taken place unnoticed, disappearing as soon as the shearing forces cease to act. In cases where the shearing stresses actually have been excessive, relief may have been obtained by local shearing of rivets in the horizontal seams, resulting in greater stresses being thrown on the flanges of the ship girder. Such shearing of the rivets in the longitudinal seams in the region of the neutral axis is not uncommon and has led to the adoption of triple-riveted seams in certain classes of ships in the region most exposed to shear stresses.

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* "Applied Elasticity," by S. Timoshenko and J. M. Lessells, p. 292.

† "Structural Design of Warships," E. and F. N. Spon, Lond., 1915, p. 27.

Example 2.—It is of interest to calculate the buckling stress in shear for the same panel as considered in Example 1. From Table 9, α_1 will be equal to about 6.0 and since all the other elements are the same as in case of buckling by compression, in which $\alpha_1 = 15.8$, the critical buckling stress in shear is,

$$p_{sr} = \frac{6.0}{15.8} \times 4\,400 = 1\,670 \text{ lb. per sq. in.} = \tfrac{3}{4} \text{ ton per sq. in.}$$

Actually, the maximum calculated shearing stress may reach about 4 tons per sq. in., so that buckling of the plating would seem very likely to occur; but, again, the boundary conditions are probably more favorable than was assumed.

Local buckling and wrinkling are known to have occurred in the docking of ships, both in floor-plates and in the lower parts of bulkheads when excessive pressures have existed on certain docking blocks. The stresses are not readily calculated in such cases on account of the complexity of the structure; furthermore, the pressure on the blocks is difficult to determine. The writer believes that naval architects can learn a great deal from civil engineers, but he believes also that civil engineers could obtain much valuable information by an analysis of shipbuilding practice. Evidently, civil engineering lends itself much better to the application of scientific methods than naval architecture. In ships, the straining actions are far more uncertain and varied in character than in stationary structures, and dynamic forces of great magnitude occur quite frequently.

DR. ING. A. NADAI* (by letter).—Questions concerning the determination of buckling loads in structural parts subjected to a somewhat complicated system of lateral or axial forces formerly have seemed to be insolvable. In a few cases it was possible to calculate the critical loads under which the equilibrium of an elastic system becomes unstable, but only when the differential equation for the lateral deflection of the structural part could be integrated in finite terms. In many cases it was not possible to express the solution or the integrals of the differential equation by means of known functions. Many problems of practical importance of this kind could be mentioned.

If one inquires more closely, it will be found that in such cases the engineer is not greatly interested in the actual form of the function which solves his differential equation. In other words, it is not so important for him to establish the complete integral of the differential equation of his problem or to determine a special integral satisfying given boundary conditions. All he wishes to know is the lowest value of the critical load which will produce buckling.

From a mathematical standpoint the answer to such questions involves the determination of a certain parameter or constant which already enters into the differential equation. With the ordinary methods this constant, or the "critical load", cannot be determined until the form of the solution has been found, that is, until the differential equation has been integrated for the given boundary conditions. This requires, in most cases, the solution of a transcendental equation and the determination of its roots.

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In such cases the methods used by Professor Timoshenko have proved successful. As he has shown in a variety of examples of practical importance, it is not at all necessary to write down the differential equation for the deflection of the beam and to search for its integrals. By using certain expressions containing the potential energy of the system of external forces and the energy of elastic distortion, it is possible to calculate an approximate value of the buckling load, without knowing the more exact form of the elastic line of the distorted structural part. Instead of the differential equation and its integration, an energy principle is used to express the condition of stability of the equilibrium of the system. In the case of stable equilibrium of an elastic system, the total sum of the potential energy of the external forces and of the elastic distortions must be an extreme value and a minimum. In other cases, as, for example, in a system which is under buckling load, this sum remains unchanged for certain small, special "variations" of the form of the equilibrium of the elastic bar. Often this energy sum can be treated for the case of the buckling of a system the same as if the system were in a condition of stable equilibrium, that is, by searching simply the form of a function (the elastic line) for which a certain definite integral (the expression of the energy sum) becomes an extreme value.

Perhaps it has not yet been noticed by engineers that, for a given differential equation of a problem, there exist an infinite number of such equivalent conditions, that is, of such definite integrals that have extreme values. From any of these definite integrals, the unknown function can be derived. Consider, for example, the linear differential equation of the second order of a function, $y = f\,(x)$:

$$y'' + \alpha\, y = 0 \quad \ldots\ldots (32)$$

which should be integrated under the condition that, for $x = 0$, $y' = 0$, and for $x = l$, $y = 0$. The integral is, obviously,

$$y = a \cos x \sqrt{\alpha} \quad \ldots\ldots (33)$$

Under these conditions the integration constant, a, in Equation (33), is undetermined and, because of the condition that when $x = l$, $y = 0$, the parameter, α, must be found so that $\alpha\, l^2 = \frac{\pi^2}{4} = 2.46740$; but $\alpha\, l^2$ can also be chosen so that it becomes equal to certain multiples of the quantity, $\frac{\pi^2}{4}$. As is well known, the elementary problem is to determine the lowest load, P, under which a straight bar of length, l, clamped at one end, loaded centrally at the other, and allowed to deflect laterally, would lead to Equation (33). The critical load is found by Euler's formula:

$$\alpha\, l^2 = \frac{P\, l^2}{I\, E} = \frac{\pi^2}{4} \quad \ldots\ldots (34)$$

in which, P is the load; I, the moment of inertia of the buckling bar; E, the modulus of elasticity; and l, the length of the bar.

The same problem, treated as one of calculus of variation, would require the determination of the function, y, for which the definite integral, $\int_0^l [(y'')^2 - \alpha\, (y')^2]\, d\, x$, becomes an extreme value. This integral contains the sum of the potential energy of the external force, P, and a, the elastic energy of bending.

Another possibility would consist in searching for an unknown function, $y = f\,(x)$, from another condition as follows: Assume that y is replaced by some approximate function, $\eta = \phi\,(x)$, which could contain some constants, c_1, c_2, c_3, etc. The second derivative of this function and the expression, $\Delta = \eta'' + \alpha\, \eta$, could then be computed.* If η and α were the exact values of y and α, then Δ should become 0 for every value of x. In general, this will not be the case, the approximation being closer as the values of Δ, or of the integral, $\int_0^l \Delta^2\, d\, x$, become smaller. A sum of squares can never become negative and, therefore, the best approximation for η and α is obtained by determining the constants, c_1, c_2, c_3, etc., that enter into the value of η so as to make the integral a minimum. As seen from this, another requirement for the function, y, and the parameter, α, would consist in determining them from the condition that the definite integral of the squares of the quantity, Δ, should become a minimum, or:

$$\int_0^l \Delta^2\, d\, x = \int_0^l (\eta'' + \alpha\, \eta)^2\, d\, x \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

A third possibility would be as follows: Equation (32), if written in the form, $y'' + \alpha\, y = 0$, represents the equation of a simple harmonic motion of an oscillating material point. The independent variable is now the time, t, and y is the distance traveled by the point.* The constant, α, contains the mass and the restoring force. Substitute t for x in Equation (33); then, $y = a \cos t \sqrt{\alpha}$; but y can now be determined also from another condition, namely, that the definite integral, $\int_0^{\frac{T}{4}} [(y')^2 - \alpha\, y^2]\, d\, t$, should become an extreme value. The integral contains the difference of the kinetic and potential energy of the moving point, the upper limit, $\frac{T}{4}$, now contains the period, T, of an oscillation.

If the relation, $y = a\left(1 - \frac{x^2}{l^2}\right)$, is used instead of the exact solution expressed in Equation (33); if $0 < x < l$; and if y is determined from the condition that the integral, $\int_0^{\frac{T}{4}} [(y')^2 - \alpha\, y^2]\, d\, t$, or $\int_0^l [(y')^2 - \alpha\, y^2]\, d\, x$, should become an extreme value, this leads to the conclusion that $\alpha = 2.50$. From the condition that $\int_0^l [(y'')^2 - \alpha\, (y')^2]\, d\, x$ must become an extreme value, it is found that

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* y', y'', etc., designate the derivation of y with respect to the time, t.

$\alpha = 3$. As seen from this, derived from the first condition, a value of α is found which is only greater by 1.3% than its exact value, $\alpha = \frac{\pi^2}{4} = 2.46740$, while the second condition would give a greater error of about 20 per cent.

As these examples show, there exists a variety of equivalent expressions (forms of definite integrals) from which the values of critical loads under buckling or—in the corresponding case of dynamics—of the frequencies of vibrating systems can be derived. Those definite integrals which contain derivatives of the lowest order of the unknown function will probably give the values of the critical speed or the critical load with greater accuracy.

It would perhaps be interesting to discuss some of the equivalent expressions of the type mentioned herein, which could be used for the purpose of determining the values of buckling loads. The writer does not know how far such questions have been treated.

Finally, another example will illustrate the usefulness of the method given by Professor Timoshenko. Consider a circular plate of radius, $r = a$, clamped along its circumference and exposed to a constant pressure, p, acting radially in the plane of the plate. The critical value of the pressure that will buckle the plate is given by solving the differential equation:

$$r^2 \phi'' + r \phi' + \left(\frac{p r^2}{N} - 1\right) \phi = 0 \quad\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

for the slope, ϕ, of the deflected middle surface of the plate. In Equation (36), N denotes the plate rigidity, or,

$$N = \frac{E h^3}{12 (1 - \nu^2)} \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

in which, E is the modulus of elasticity; h, the thickness of the plate; ν, Poisson's ratio; and r, the radial co-ordinate. The solution is found by Bessel's function:

$$\phi = c\, I_1 \left(\frac{\lambda r}{a}\right) \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

in which, $\lambda^2 = \frac{p a^2}{N}$, and for λ any root of the equation, $I_1 (\lambda_n) = 0$, can be taken.* In Equation (38), a is the radius of the plate. The first positive root of the last equation is $\lambda_1 = 3.8317$. The corresponding value of the lowest buckling load is $p = \frac{\lambda_1{}^2 N}{a^2} = 14.682 \frac{N}{a^2}$.

By using the relations, $x = \frac{r}{a}$, $y = \phi \sqrt{x}$, and $\lambda^2 = \frac{p a^2}{N}$, Equation (36) can be easily transformed into the form:

$$y'' + \left(\lambda^2 - \frac{3}{4 x^2}\right) y = 0 \quad\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

For a corresponding definite integral, $\int_0^1 \left[(y')^2 - \left(\lambda^2 - \frac{3}{4 x^2}\right) y^2\right] d x$, can be taken, which is now to be made an extreme value. When $y = c\,(x^3 - x) \sqrt{x}$, this integral gives the first approximate value of the parameter, $\lambda_1^2 = 16$, with an error of 9 per cent.

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* The solution seems to have been established first by Bryan; see, also, A. Nadai, "Die elastischen Platten," Springer, Berlin, 1925, p. 249.

By using a better approximation, $y = c_1 (x^3 - x) \sqrt{x} + c_2 (x^5 - x) \sqrt{x}$, the result is $\lambda_1^2 = 14.702$. When compared with the exact value, $\lambda_1^2 = 14.682$, it can be seen that this is a very good approximation for the buckling load of the plate.

S. TIMOSHENKO,* ESQ. (by letter).—The purpose of the paper was to give a brief discussion of several problems of elastic stability, different from simple column problems and of interest to structural engineers. In all cases it was assumed that the bars were slender and the plates thin, so that buckling occurred within the elastic limit of the material. There are many cases of more complicated conditions in which buckling occurs beyond the elastic limit, and the writer completely agrees with Professor Westergaard's statement that these problems "represent a field for future research, both experimental and analytical."

In the case of trusses, the compressed members are usually in a condition of eccentric compression, due to rigid joints. The magnitude of the eccentricities can be calculated by the methods used in the analysis of "secondary stresses" in trusses. Having these eccentricities, it seems logical to design these members as bars submitted to the simultaneous action of compression and bending. As an example, take the simplest case, when the eccentricities on both ends are equal and in the same direction. The maximum bending moment at the middle is then given by the well-known secant formula:

$$\mu_{\text{max.}} = P_e \sec \frac{l}{2} \sqrt{\frac{P}{E\,I}} \qquad \text{(40)}$$

in which, P is the eccentrically applied compressive force, and e, the magnitude of the eccentricities.

The maximum compressive stress is:

$$s_{\text{max.}} = \frac{P}{A} + \frac{\mu_{\text{max.}}}{S} = \frac{P}{A}\left(1 + \frac{e}{r} \sec \frac{l}{2} \sqrt{\frac{P}{E\,I}}\right) \qquad \text{(41)}$$

in which, $r = \frac{S}{A}$.

In using Equation (41) to determine cross-sectional dimensions, it is important to note that the maximum stress is not proportional to the load, P, but increases more rapidly than the load. Hence, it is not logical in Equation (41) simply to make $s_{\text{max.}}$ equal to the working stress, s_w, when designing the bar. This is explained by Fig. 18 which shows $s_{\text{max.}}$ as a function of P, as given by Equation (41). This curve has as its asymptote the vertical line, $B\,C$, such that,

$$O\,B = P_{cr} = \frac{E\,I\,\pi^2}{l^2} \qquad \text{(42)}$$

Select a working stress that is a certain amount less than the yield-point stress, s_y, for example, let $s_w = \frac{s_y}{2}$. In Fig. 18, this corresponds with the point

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D, on the curve and it follows that the load is P_w. It will be seen that, due to the non-linear relation between $s_{\text{max.}}$ and P, the ratio, $\frac{P_w}{P_y}$, is different from the ratio, $\frac{s_w}{s_y}$.

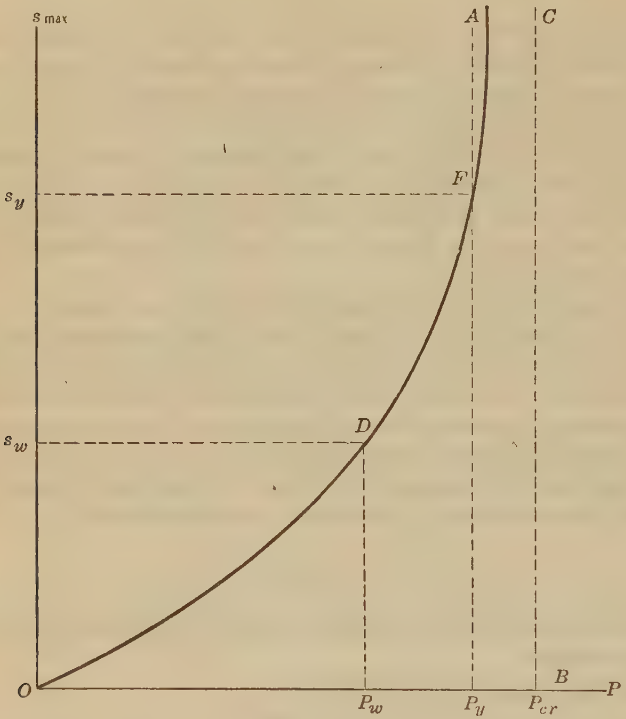

FIG. 18.

While s_w is one-half s_y, P_w is more than one-half P_y, and the factor of safety with respect to the beginning of yielding is smaller than 2. In order to arrive at the prescribed factor of safety, let s_w and s_y represent, respectively, the working stress for short compressed bars and the yield point of the material, and let n be the factor of safety. This factor, then, must be such that the maximum stress, given by Equation (41), will equal the yield point when the load, P, is increased n times. It is assumed that Equation (41) is accurate enough to describe conditions also when $s_{\text{max.}}$ approaches s_y. For this limiting condition, Equation (41) becomes:

$$s_y = \frac{n\,P}{A}\left(1 + \frac{e}{r}\sec\frac{l}{2}\sqrt{\frac{n\,P}{E\,I}}\right) \ldots\ldots\ldots\ldots\ldots\ldots (43)$$

from which,

$$\frac{1}{n}\,s_y = s_w = \frac{P}{A}\left(1 + \frac{e}{r}\sec\frac{l}{2}\sqrt{\frac{n\,P}{E\,I}}\right) \ldots\ldots\ldots\ldots (44)$$

Comparison of Equation (44) with Equation (41) shows that the replacement of P by nP, under the radical, takes proper account of the fact that $s_{\text{max.}}$ increases faster than P.

If P, e, n, and s_y are known, the cross-sectional dimensions may be determined by cut-and-try methods so as to satisfy Equation (44). Manipulation with this equation can be greatly simplified by preparing tables for $\frac{P}{A}$ for various values of n and $\frac{e}{r}$ and for various slenderness ratios of struts. Such tables were calculated for several cases by K. S. Zavriev* and an analogous method was proposed by A. Ostenfeld.†

An analogous analysis can be developed for any other condition of eccentricity. With the tables mentioned, compressed members with any given factor of safety can easily be designed. This method of treating compressed members also gives a logical basis for dimensioning lattice-bars of built-up columns. These bars must be strong enough to sustain the shearing force, the value of which is easily derived when the eccentricities in the application of the compressive force are known.

Both methods of reasoning applied by Professor Hovgaard in deriving Equations (27) and (28) are legitimate and both of them reach the same final result. This subject was discussed elsewhere in detail by the writer‡ and he did not consider it necessary to repeat it.

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* *Bulletins*, Soc. of Engrs. of Technology, 1924, St. Petersburg.

† *Ingeniorvidenskabeligeskrifter*, 1929, Copenhagen.

‡ *Zeitschrift für Math. u. Phys.*, Vol. 58, 1910, p. 337.

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Paper No. 1750

PRE-DETERMINING THE EXTENT OF A SEWAGE FIELD IN SEA WATER*

By A. M. Rawn,† and H. K. Palmer,‡ Members, Am. Soc. C. E.

With Discussion by Messrs. George W. Fuller, Kenneth Allen, R. F. Goudey, Warren E. Howland, Harrison P. Eddy, and A. M. Rawn and H. K. Palmer.

Synopsis

This paper presents a method for pre-determining the probable area and extent of a sewage field in sea water, when the quantity of sewage flow and the direction and depth of discharge below the ocean surface are given. The formulas may also be applied to define the probable limits of a sewage field at any given degree of dilution.

Research by the writers fails to reveal any established numerical relationship between the known or pre-determined factors surrounding an ocean outfall outlet and the probable amount of dilution obtained at the ocean surface over the outlet or of the probable spread of the field. In many instances bacteriological surveys made of the area of ocean pollution at sewer outlets have disclosed the extent of the permissible pollution to be far from what the designers anticipated. The predicted area in such cases was based largely upon an "acres-per-second-foot-of-sewage" factor used as a constant rather than as a variable, dependent on the elements of design and conditions existing at the outfall site.

Health standards, as related to shore waters along ocean beaches, define the limits to which pollution of such waters may be carried without creating a menace to health or a public inconvenience. These standards are often found

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to be maintained by frequent reconstruction of the ocean outlet following quarantine of the adjacent shore waters. By means of the formula presented herewith, an attempt is made to eliminate some of the elements of doubt and conjecture regarding the probable spread of a sewage field and to aid in the design of ocean outlets which will not of necessity be abandoned or reconstructed prior to the expiration of their anticipated useful life.

The particular factor that influenced the undertaking of this series of experiments is the contemplated construction of an ocean outfall by the Los Angeles County Sanitation Districts on the California coast line, south of the City of Los Angeles. The information and data assembled during the series of experiments, and the formulas derived from the experimental and research work, are presented for the information of those engaged in similar work or studies.

NOTATION

The following notation has been used in the paper:

a = constant in the equation of the path from a horizontal outlet.
A = area, in square feet, of the cross-section of the rising column of water at any level (y) under observation.
B = constant.
D = diameter of the outlet, in inches.
E = horizontal distance from the center of the rising column to the edge of the sewage field.
F = constant used in determining velocity in the sewage field.
K = constant.
L = length of path between outlet and surface.
n = number of outlets.
P = thickness of sewage field.
Q = quantity of sewage or fresh water.
S = dilution factor.
S_0 = dilution factor at head of rising column.
t = time, in seconds.
U = velocity of ocean current.
V = velocity of the fresh-water jet, or of diluted sewage in the sewage field.
x = horizontal distance from the outlet to the center of the rising column at any level, y, under consideration; or the horizontal distance from the center of the rising column at the surface to any point in the field.
X_0 = radius of virtual ring from which the sewage field spreads.
X = horizontal distance from the outlet to the center of the rising column at the surface (see Fig. 1).
y = vertical distance from the outlet to the point in the center of the rising column that is under consideration.
Y = total depth of salt water above the outlet.

Early Observations of the Behavior of a Fresh-Water Jet in Salt Water

From preliminary studies and minor experiments with small colored fresh-water jets about $\frac{1}{4}$ in. in diameter, discharging less than $\frac{1}{2}$ gal. per min. in clear sea water, the following behavior was indicated. When discharged vertically upward from a submerged outlet a jet was seen to rise very much in the form of a dense cloud of smoke; it rolled and billowed in much the same manner, taking finally a shape which approximated an inverted cone with its base resting at the surface of the water and its apex at the point of discharge. Repeated trials indicated that the cone was uniformly constant in dimension and appearance for like conditions of discharge.

From a jet of fresh water discharged horizontally below the surface of the salt water the path of the fresh water appeared to approximate that of a projectile, except that it curved upward and was retarded by the turmoil occasioned by the billowing of the fresh water in the same manner as smoke billowing from a stack. A nozzle was fixed at a depth such that, when the jet was discharged vertically upward, it just reached and broke the surface of the salt water. The writers demonstrated that the same quantity of water from the same nozzle, discharged horizontally at the same depth, would not reach and break the surface of the salt water as a well-defined jet. This indicated that, other conditions being equal, the absorption of sea water into the sewage—or the degree of intermingling of the two waters—was greater for a horizontal than for a vertical discharge outlet.

With the jet pointing vertically downward, the path of fresh water gave the appearance of an inverted fountain with the head fluctuating or varying slightly in elevation. The fresh water traveled, sometimes, all to one side of the jet and frequently, in its upward path, completely surrounded the downward jet. The upward path then took the same approximate form as that from a vertical discharge line, except that it was not quite so concentrated in color and for a like amount of discharge presented a somewhat larger ring as it broke the surface. For a jet inclined 45° upward, conditions appeared to be about an average between that for a vertical and a horizontal discharge.

A cap in the form of a funnel with the spout closed, was lowered over the vertical jet until the edge of the funnel was below the point of outlet. The fresh water then gave the appearance of passing over an inverted weir, the upward path from the edges of the weir being about the same form as the rising column from a vertical jet, but with greater diffusion of fresh water into salt water and a larger field where the fresh water broke the surface of the salt water.

The observation of the behavior of the streams of fresh water from the miniature jets in the salt water formed the basis for the design of an apparatus to be used in the subsequent studies. It was determined from these meager observations that it would be necessary to be able to measure actual dilution at the surface, as produced under varying conditions of depth, quantity, velocity, and direction of discharge.

Description of Methods and Appliances

The first observations were made in tubs of salt water and in the quiet lagoons at Alamitos Bay, California. They indicated little that was conclusive because no method was provided for measuring the value of the dilution factor at the time the stream reached the surface of the salt water or during its spread over that surface.

The site chosen for carrying the experiments to a conclusion was a basin in Los Angeles Harbor about 2 acres in extent, protected on two sides by land and, with the exception of a small entrance, on the other two sides by fairly tight sheeted wharves. Few steamers passed the entrance, and the only other disturbance due to wave action was occasioned by fishing boats and yachts brought into the basin for repairs. Equipment was mounted on a large raft moored in the basin which permitted the experiments to be made without reference to the tides. Depths of 13 ft. were obtained at high tide with 9 ft. possible at all times. The arrangement is shown in Fig. 1.

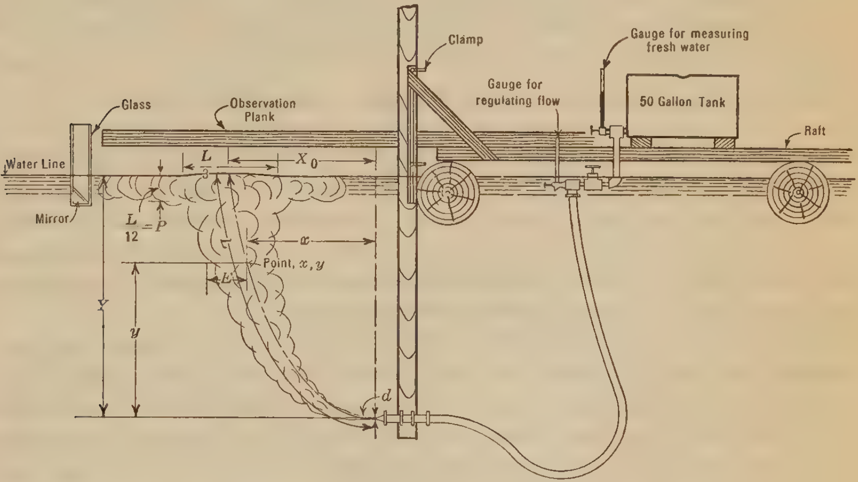

Fig. 1.—Experimental Apparatus.

A fresh-water tank (a 50-gal. oil drum), was mounted on its side on the raft; the contents could be discharged through a 1½-in. flexible hose, 12 ft. long. Pipe couplings, nipples, and reducers, could be attached to the end of the hose, permitting discharge through varying sizes of nozzles, which were ordinary pipe nipples ranging from ¼ in. to 1¼ in. in inside diameter. A few experiments were performed with 1½-in. and 2-in. nozzles, with a section of 2½-in. fire hose as a discharge line. The permanent end coupling on the hose was attached to a 1-in. by 6-in. board, 16 ft. long, which could be clamped to an upright post on the raft with the nozzle at any desired depth.

Flow from the drum was controlled by a globe valve located about a foot below the lowest elevation of the drum. Thus, the variation in flow due to change in head, as the drum emptied, was minimized. As a check on the uniformity of flow a gauge glass was placed in the line below the valve, and was kept, as nearly as possible, at a constant head by adjusting the valve opening from the drum.

The drum was carefully calibrated and the rate of flow during any one experiment was determined by measuring the quantity of water discharged during that experiment. Time was measured to the nearest second and the fluid surface drop in the tank was indicated by the fall in a gauge glass in the outlet just above the globe valve.

Eosine Y was introduced into the fresh water as a coloring agency, the proportions being about ½ grain per gal. for shallow-water experiments when resulting low values of S were to be measured and as much as 2 grains per gal. for either deeper experiments or those in which the field spread was to be measured.

The value of S at any time was measured by diluting with sea water a measured sample of the tank fluid until it compared in color with the sample taken from the sewage field. Thus, if 10 cu. cm. of the fluid taken from the tank were diluted until it matched in color a sample taken from the jet or field and the sample was then found to contain 120 cu. cm. of combined colored fresh water and sea water, the resulting S was $\frac{120}{10} = 12$.

Samples were compared in 8-oz. oil-sample bottles, 1½ in. in diameter and 6 in. long, and the high dilutions were matched by holding the bottles, base downward, over a mirror so as to get a maximum concentration of color by looking indirectly through the bottle along its long axis. It was found that differences in dilution could be measured in this manner to a value of 150 for S with a probable error of about 4 per cent.

Conditions below the surface, particularly those which marked the plane between the clear salt-water and the colored fresh-water field, were investigated by right-angle reflection. The apparatus consisted of an upright oblong box closed at the bottom and open at the top. One side of the box was fitted with a glass pane, and a mirror was set in the bottom at an angle of 45° with the glass side. The possibilities of such an instrument are apparent.

The paths of the smaller streams of fresh water were found to be subject to some variation as a result of slight currents in the sea water, and in order to measure the horizontal co-ordinate of the path of a horizontal jet with a minimum of possible error, a double jet was placed at times on the nozzle. It consisted of a "tee" and the two jets discharged horizontally in opposite directions. The distance between the centers of the two rising columns was measured at the surface; from this was subtracted the distance between the two nozzle ends and the remainder was divided by 2, giving the required distance without reference to variation in the path which was the same for both jets. Interference between two rising columns below the surface and the spread of a double field was investigated with the same apparatus slightly modified.

The spread of the field on the surface was determined by observing the time when the edge of the field passed fixed points along the edge of the raft and along a plank cantilevered over the water at right angles to that edge.

As a check on results obtained from the foregoing experiments and in order to fix with greater accuracy the upper limits of the curves, values of *S* were determined for four known outlets in the vicinity of Los Angeles, namely, those at Los Angeles Hyperion Outfall, Long Beach, Santa Monica, and Santa Barbara. One at Catalina Island was investigated, but the sewage was found to be so diluted on reaching the water surface that samples taken would not yield results.

In the existing outfall investigations the salt content of each sample was measured. A solution of silver nitrate was prepared and standardized by titrating 2 cu. cm. of ocean water with it; 2 cu. cm. of each of the samples was then titrated in the same manner and when there was sufficient salt in the raw sewage to effect results, it also was titrated. By this means values of *S* up to 35 or 40 could be determined with accuracy.

To determine the thickness of a sewage field the difference in electrical conductivity of sea water and diluted sewage was noted with a Wheatstone bridge. The galvanometer was adjusted with the electrodes on both arms of the bridge immersed in clear salt water well below the layer of sewage. On raising one arm of the bridge the galvanometer would give a kick as the electrodes entered the diluted sewage. A dilution of 25 could be detected in this way.

TABLE 1.—NUMBER OF OBSERVATIONS ON VERTICAL JETS.

Depth, in feet.	DIAMETER OF NOZZLE OPENING, IN INCHES.					Total number of observations.
	⅜.	½.	¾.	1.	1¼.	
1.2	....	3	3	3	3	12
1.5	10	3	7	3	10	33
2.5	3	3	3	3	3	15
3.5	3	3	3	3	3	15
4.5	4	3	3	3	3	16
5.5	11	3	17	3	14	48
6.5	3	3	3	3	3	15
7.5	3	3	3	4	3	16
8.5	....	3	3	3	3	12
9.5	7	....	11	3	16	37
10.5	....	....	....	3	....	3
Total.......	44	27	56	34	61	222

VERTICAL JETS

It was apparent that there were fewer factors affecting the dilution from vertical jets than from other types, so they were studied first. A series of 222 observations was made at depths ranging from 1.2 to 10.5 ft. and using $\frac{3}{8}$-in., $\frac{1}{2}$-in., $\frac{3}{4}$-in., 1-in., and $1\frac{1}{4}$-in. nozzles. The number of experiments at various depths for each size of outlet are shown in Table 1.

If for any fixed depth the results of each experiment are plotted with values of Q, in gallons per minute, as ordinates and S_0 as abscissas, the equation of the curve takes the form:

$$S_0\, Q^{0\cdot67} = f\,(y) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and this constant is the same for all sizes of nozzles. Subsequent comparison with results at the existing Long Beach Outfall while 3 000 gal. per min. of sewage was being discharged at a depth of 9.5 ft. and where the resulting S_0 was small, showed that the equation took the form:

$$(S_0 - 1)\, Q^{0\cdot61} = f\,(y) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

The factor $(S_0 - 1)$ is used instead of the factor, S_0, in order to avoid the anomalous case arising from the discharge of a large quantity of sewage at shallow depth, under which condition the value of S_0 might become less than unity, which would correspond to a concentration of the sewage. Since $(S_0 - 1)$ indicates the number of units of sea water per unit of sewage, there is a logical reason for its use.

In Fig. 2 results are plotted at the three depths at which the major number of experiments were taken, and the result of the experiment described at Long Beach is also shown.

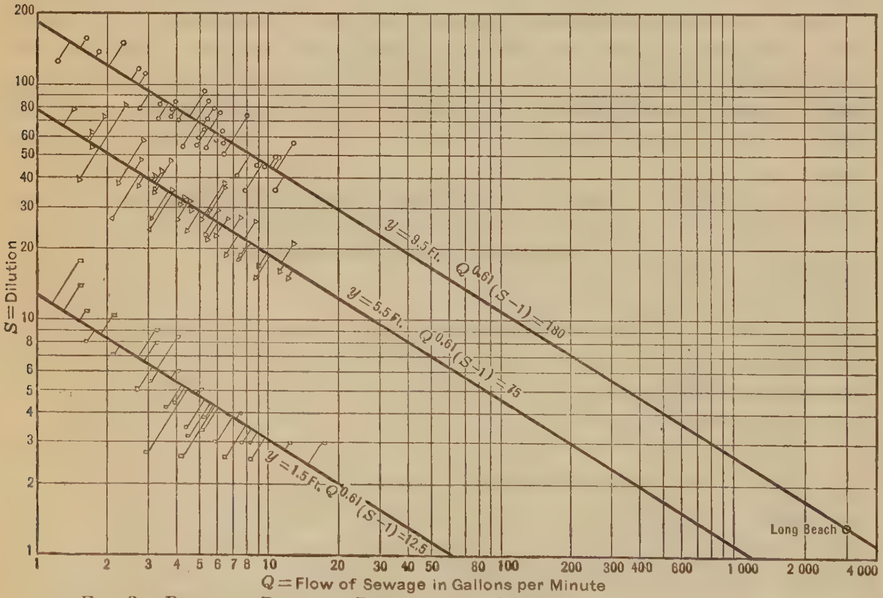

FIG. 2.—RELATION BETWEEN DILUTION AND QUANTITY AT CONSTANT DEPTHS.

HORIZONTAL JETS

This type of discharge is complicated by the fact that the rising column follows a path similar to that of a projectile, except that it curved upward. The direction of the path is modified by the inertia of the salt water picked up by the rising column, the tendency being to reduce its horizontal component. A total of 388 experiments was made, using horizontal jets, the numbers at each depth, with different sizes of nozzles, being shown in Table 2.

It was apparent from the beginning that the value of S_0 was greater from the horizontal jets than from the vertical, other conditions being equal, and this group of experiments indicated that for this case Equation (1) should be written:

$$(S_0 - 1) Q^{0.61} = f (L) \ldots\ldots\ldots\ldots (3)$$

The first requisite in the solution of Equation (3) was to determine the length of the rising column between the point of discharge and the salt-water surface. This length is, in turn, dependent on the equation of the curve of the path of the rising column. In order to avoid, as far as possible, error introduced by lateral currents in the sea water, the horizontal co-ordinate, X, was measured with the use of the double nozzle heretofore described.

TABLE 2.—NUMBER OF OBSERVATIONS ON HORIZONTAL JETS.

Depth, in feet.	DIAMETER OF NOZZLE OPENINGS, IN INCHES.								Total number of observations.
	¼.	⅜.	½.	¾.	1.	1¼.	1½.	2.	
0.5	7	7	5	....	....	....	....	....	19
1.	....	....	....	7	....	....	....	....	7
1.3	7	7	9	....	....	....	....	....	23
1.5	....	....	....	....	3	3	....	....	6
2	7	7	7	7	3	8	7	5	51
3	7	7	7	7	3	8	7	....	46
4	7	7	7	7	3	8	7	7	53
5	....	8	7	7	3	8	7	....	40
6	....	....	7	7	3	8	7	7	39
7	....	....	....	....	3	8	7	....	18
8	....	....	....	....	3	8	7	6	24
9	....	....	....	....	3	10	7	....	20
10	....	....	....	....	....	8	6	7	21
11	....	....	....	....	....	....	....	7	7
12	....	....	....	....	....	....	....	7	7
13¼	....	....	....	....	....	....	....	7	7
Total	35	43	49	42	27	77	62	53	388

Comparison of results using different quantities from a given size of nozzle fixed at a constant depth lead to the relationship:

$$x = K V^{\frac{2}{3}} \ldots\ldots\ldots\ldots (4)$$

in which, K is a constant for that depth.

Comparing the values of K obtained at various depths for one size of nozzle, leads to the equation:

$$K = B \sqrt[3]{y} \ldots\ldots\ldots\ldots (5)$$

in which, B is a constant for that size.

Furthermore, comparing the values of B for nozzles of different sizes, indicates that it has a progressive increase expressed in the equation:

$$B = \sqrt[3]{D} \ldots\ldots\ldots\ldots (6)$$

Combining Equations (4), (5), and (6), and substituting for $\sqrt[3]{V^2 D}$, the constant, a, Equation (4) becomes,

$$x = a \sqrt[3]{y} \ldots\ldots\ldots\ldots (7)$$

which is a cubical parabola with the origin at the nozzle.

The formula for the length of this path is too cumbersome to use in practice, but the two following approximations will give the length with sufficient accuracy:

Case 1, when $\frac{Y}{X} > \frac{1}{3}$,

$$L = Y + (0.8a)^{\frac{3}{2}} - \frac{0.1685\, a^2}{\sqrt[3]{y}} \qquad \text{(8)}$$

Case 2, when $\frac{Y}{X} < \frac{1}{3}$,

$$L = X \left\{ 1 + 0.9 \left(\frac{Y}{X}\right)^2 \right\} \qquad \text{(9)}$$

The exact formula and the two approximations (Equations (8) and (9)) were developed by Mr. Gordon Ward, of the California Institute of Technology.

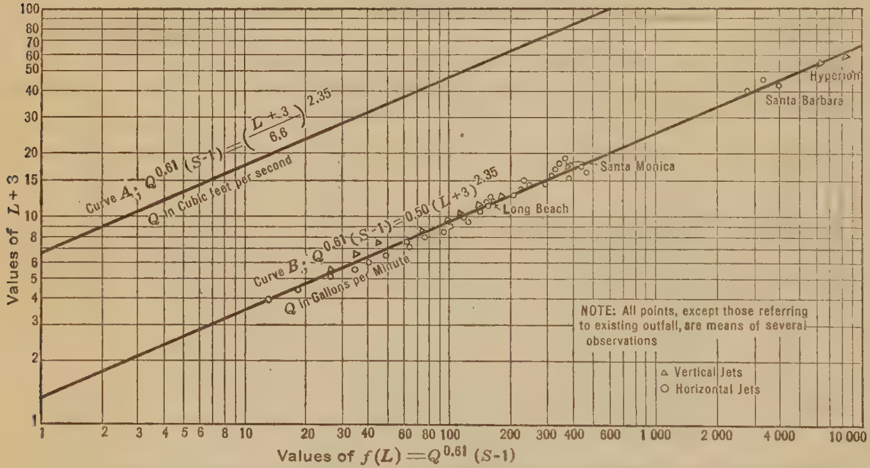

FIG. 3.—RELATION BETWEEN DILUTION AND LENGTH OF PATH.

RELATION OF DILUTION, S_0, TO LENGTH OF PATH, L

Equation (2) for vertical jets becomes identical to Equation (3) for horizontal jets if L is substituted for y. A value for $f\ (L)$ was computed for each horizontal and vertical experiment by means of Equation (3) and was plotted with $f\ (L)$ as the abscissa and L as the ordinate. In Fig. 3 the results of all values for $f\ (L)$ for any one depth were averaged, and the resultant was plotted as a single point on the curve. With horizontal jets the length of path being a complex function of the depth and velocity, it was impossible to regulate any experiment so that the length, L, would equal a pre-determined value. Therefore, instead of the observations being segregated into groups of single lengths as with vertical jet experiments, they were averaged within a narrow range of L values. All values within a range of 0.5 ft. were combined in order to determine the average. The values for Santa Monica and Hyperion were not averaged, but were plotted directly; those for Long Beach and Santa Barbara were the resultant averages of several groups of experiments.

When first plotted, the results lay on a curve, and it was necessary to add 3 ft. to the length of path, which resulted in the equation,

$$(S_0 - 1)\, Q^{0.61} = 0.5\, (L + 3)^{2.35} \qquad (10)$$

in which, Q is measured in gallons per minute. For large discharges it is more convenient to measure Q in cubic feet per second in which case the coefficient may be readily changed.

JETS DISCHARGING VERTICALLY DOWNWARD

In some cases outfall pipes are carried on wharves and the pipe is clamped to a pile so as to discharge the jet downward some distance below the surface of the water. The fresh water then acts in a manner similar to an inverted fountain and the distance to which the water flows downward before turning up, is approximately 0.5 V. This distance is only approximate because the so-called inverted fountain is very unstable, the "head" constantly descending and rising. At Santa Monica, where this system of discharge is used, it was noted that the sewage came to the surface in waves, the flow at times almost ceasing, followed by a "crest" when a large quantity would boil up.

The resulting dilution is apparently that due to the total length of path from the end of the outlet down to the turning point and back to the surface, but in fairly large outlets it is probable that little dilution occurs on the downward path, so that the effective length would be only from the bottom of the downward jet to the surface.

JETS DISCHARGING UPWARD AT 45 DEGREES

Experiments were conducted with jets discharging 45° from the vertical and the results indicated that the length of path, L, was the mean of the lengths for vertical and horizontal jets, other conditions being equal. No formula is given for this case. However, it appears that the mean of vertical and horizontal formulas will indicate the value for S_0 for inclined jets within reasonable limits.

DIAMETER OF A RISING COLUMN AT THE SURFACE OF THE SALT WATER

In making the experiments, the time required for the jet to reach the surface was measured to the nearest second; furthermore, the diameter of the column was measured just as it reached the surface and before the field began to spread. This was somewhat indefinite as the column of water, billowing upward, was not exactly round, and it was difficult to determine a mean diameter as it broke the surface of the salt water. The average of a large number of observations, however, indicated this dimension to be approximately one-third the length of the path, L. (See Fig. 1.)

In the case of a vertical jet the quantity passing any horizontal plane is $Q\,S$ and the velocity at that level is,

$$V = \frac{Q\,S}{A} = \frac{4\,Q\,S}{\pi}\left(\frac{3}{y}\right)^2 = \frac{36\,Q\,S}{\pi\,y^2} = \frac{d\,y}{d\,t} \qquad (11)$$

Therefore,

$$t = \int_0^y \frac{\pi\,y^2}{36\,Q\,S}\,d\,y \qquad (12)$$

Since S is a complex function of y, this integration (Equation (12)) was performed mechanically. A comparison of the observed and calculated time of rising demonstrated that the observed time was approximately 0.6 of the calculated time, indicating that the assumed area, $A = \frac{\pi}{4}\left(\frac{y}{3}\right)^2$, was too great and that the virtual diameter of the column was more nearly $\frac{y}{4}$ for vertical jets, or $\frac{L}{4}$ for horizontal jets.

By virtual diameter is meant the diameter of a theoretical column with uniform velocity at all points in any given cross-section, and of such size that the time of rise will be equal to the observed time. In actual cases the outer parts of the column contain rolling water with an upward velocity relatively slow as compared with the center of the column.

For large sewers the diameter of the nozzle should be added to $\frac{y}{4}$ or $\frac{L}{4}$ to obtain the true diameter of the column. The importance of this added factor varies inversely with the depth of discharge below the water surface.

THICKNESS OF THE SEWAGE FIELD

As the dilute sewage in the rising column reaches and breaks the surface of the salt water its tendency is to spread laterally in all directions over the heavier salt water. The plane of contact between the salt water and the lighter mixture of salt and fresh water is not clearly defined, owing to the natural tendency of the two fluids to intermingle. The average of many observations indicates a value of $\frac{L}{12}$ for the field thickness and that this thickness remains uniformly constant over the entire apparent field (see Fig. 1). Measurements of the thickness of the fields at the Long Beach and Los Angeles Outfalls confirmed this value.

As the column of sewage rises and mixes with salt water, the velocity of the particles is progressively retarded and the straight line motion is converted into a rolling motion similar to a large column of smoke, the rolling masses gradually becoming larger until they reach the center of the column. On account of this motion of the masses there is no abrupt change in the direction taken by the individual particles where the column breaks through the surface, although the center of the masses will change direction.

The vertical velocity at the top of the rising column is,

$$V = \frac{Q\,S_0}{\frac{\pi}{4}\left(\frac{L}{4}\right)^2} \dots\dots\dots\dots\dots\dots \text{(13)}$$

In still water the area of the annular cross-section, A, of the field at any distance less than E from the center of the column (see Fig. 1), will be represented by the formula,

$$A = 2\,\pi\,x\,P \dots\dots\dots\dots\dots\dots \text{(14)}$$

The virtual diameter at the top of the rising column is $2\,X_0 = \frac{L}{4}$, and the lateral velocity at the head of the rising column will be represented by,

$$V = \frac{Q\ S_0}{\pi\,P\left(\frac{L}{4}\right)} \qquad\qquad (15)$$

Assuming that no loss of energy occurs in the change from vertical movement in the rising column to horizontal movement as the field spreads, by Equation (13) and Equation (15),

$$\frac{Q\,S_0}{\frac{\pi}{4}\left(\frac{L}{4}\right)^2} = \frac{Q\ S_0}{\pi\,P\left(\frac{L}{4}\right)}$$

or,

$$P = \frac{L}{16}$$

This indicates that the observed velocity of the horizontal motion in the field spread is less than the theoretical value, the latter being determined by assuming no loss in energy as vertical motion is changed to horizontal. Increased turbulence at the column head indicates energy loss at that point, and it is believed that the ratio, 12 : 16, may be fairly representative of such loss. Therefore, the value for P is assumed to be (see Fig. 1),

$$P = \frac{L}{12} \qquad\qquad (16)$$

SIZE OF THE SEWAGE FIELD

As the fresh water spreads the dilution continues, but since the turbulence is much less than in the rising column the rate of dilution diminishes correspondingly. It was observed that the colored fresh water moved over the salt water in concentric waves, each wave being followed by an interval of apparently clear salt water. These waves continued for some distance from the head of the rising column, and gradually merged to form a field of uniform color. This fact is noted to illustrate the difficulty of securing a single representative sample. Those taken from the same location in the field at different times, other conditions being equal, indicated that the dilution remained constant, within the limits of error of observation, at that location.

Experiments made with a view to obtaining the progressive increase in dilution from the head of the rising column to the edge of the apparent field, did not yield results that were as consistent as those indicating the dilution over the rising columns. This was probably due to the manner in which the sewage left the column head in waves. However, such experiments did show progressive increase in dilution as the distance from the head increased, and the time required for the sewage to travel a given distance was exceedingly uniform for like conditions. The latter was found to be expressed by the equation,

$$t = \frac{2}{3\,F}\,x^{\frac{3}{2}} \qquad\qquad (17)$$

Differentiating Equation (17),

$$\frac{d\,x}{d\,t} = \frac{F}{\sqrt{x}} = V \quad \text{(18)}$$

The quantity of diluted fresh water passing a ring is $Q\,S$ and the area of such annular cross-section, by Equation (14), is $2\,\pi\,P\,x$, thus the velocity is,

$$V = \frac{Q\,S}{2\,\pi\,P\,x} \quad \text{(19)}$$

Equating this value to Equation (18) and transposing terms:

$$S = \frac{2\,\pi\,P\,F\,\sqrt{x}}{Q} \quad \text{(20)}$$

At the head of the column from which the field expands, $x = X_0$ and $S = S_0$; substituting these values in Equation (20),

$$F = \frac{Q\,S_0}{2\,\pi\,P\,\sqrt{X_0}} \quad \text{(21)}$$

and substituting this value for F in Equation (20),

$$S = S_0\sqrt{\frac{x}{X_0}} \quad \text{(22)}$$

Observations of existing fields and a number of experiments indicate that $X_0 < \frac{L}{8} + \frac{D}{24}$. However, the use of this value,

$$X_0 = \frac{L}{8} + \frac{D}{24} \quad \text{(23)}$$

in Equation (22) is recommended.

In a confined channel with approximately parallel sides the cross-section of the field can increase only by thickening. The kinetic energy cannot increase and, therefore, as the velocity decreases with the increasing dilution, it is evident that the field must thicken, and, as soon as the thickness equals the depth of the water in the channel, no further dilution is possible. It is evident, therefore, that a constricted channel makes a poor location for a sewer outlet, and where it is the only available location the initial dilution should be as high as possible.

Some currents are nearly always present in the ocean, and in order to consider their effect on the field it is necessary to consider the velocity of spreading. The two particularly important considerations are the distance the field will travel directly against an ocean current and the distance it will travel directly with the current, until a certain value of S is attained.

The maximum distance the sewage will travel against the current can be expressed as,

$$e = x - x' = x - U\,t \quad \text{(24)}$$

in which, x' equals the distance the ocean current will travel in time, t, at a velocity, U, and e is the maximum distance from the center of the column head to which the sewage can travel against the ocean current. Under conditions expressed in Equation (24), $V = U$. Then, from Equation (18), $U = \frac{F}{\sqrt{x}}$, and,

$$x^{\frac{3}{2}} = \left(\frac{F}{U}\right)^{3} \dots\dots\dots\dots (25)$$

From Equation (17),

$$U\,t = \frac{2}{3}\left(\frac{F}{U}\right)^{2} = x' \dots\dots\dots\dots (26)$$

Substituting the values of x and x' from Equations (25) and (26) in Equation (24),

$$e = \left(\frac{F}{U}\right)^{2} - \frac{2}{3}\left(\frac{F}{U}\right)^{2} = \frac{1}{3}\left(\frac{F}{U}\right)^{2} \dots\dots\dots\dots (27)$$

in which, the value of e is independent of the value of S.

The distance the sewage will travel with the current will be expressed by the relation,

$$e = x + U\,t \dots\dots\dots\dots (28)$$

Mutual Interference from Multiple Outlets

The theory has been advanced in the past, and incorporated into the actual design and construction of numerous ocean outlets, that, other conditions being equal, the use of multiple outlets is advantageous.

An inspection of Equation (10) for S_0 would indicate that such an assumption is true, because the value of S_0 for $\frac{Q}{n}$, for a given depth, is greater than the value of S_0 for Q and the distance that sewage must travel to attain a definite value for S is a function of $\left(\frac{S}{S_0}\right)^{2}$.

Interference Between Rising Columns Below the Salt-Water Surface.—Two vertical outlets were attached to the discharge hose so as to permit simultaneous discharge from both at equal depth and at constant distance between centers of rising columns. As long as the depth of discharge below the surface was less than three times the distance between the two outlets, the observed and calculated values of S_0 were the same. However, when the depth of discharge below the surface exceeded three times the distance between the two outlets, the two rising columns interfered below the salt-water surface, decreasing the area of the columns in contact with pure salt water. This resulted in the observed value for S_0 being less than the theoretical for each column alone. At extreme depths, where the ratio of depth to distance between centers was great, the observed values approached the theoretical value for one outlet discharging the combined flow of the two.

A series of experiments was performed in which two horizontal jets were directed toward each other. The resulting observed values of S_0 in this instance approached very closely those for the foregoing experiment with two vertical jets at equal depths. This is reasonably explained by the fact that the two horizontal velocities directed toward each other were neutralized, the path of each was shortened, and the two columns mingled shortly after discharge and rose together as a single vertical column.

Six slots, 1 by 1-in., were cut in the rim of a can, which was inverted and placed over a vertical nozzle. This divided the flow from the nozzle into six small columns, and at shallow depths, where these columns did not interfere in rising, the resulting observed value for S_0 was that due to each small stream alone; but after interference became effective, at greater depths, the advantage was nullified.

Under the circumstances, very little horizontal velocity was imparted to the fresh water flowing through the orifices, the fresh water escaping as if it were flowing over inverted weirs with an acceleration due to the difference of the specific gravity of the two fluids and equal to $\frac{g}{40} = 0.8$.

A similar experiment was performed by inverting a can over a vertical nozzle, except that, in this instance, no slots were cut into the edge of the can. Under these circumstances the fresh water rose in the form of a hollow column, all the dilution taking place on the outer surface thereof. As the depth of discharge was increased the ratio of the observed value of S_0 with the diffuser, to the calculated value without the diffuser, decreased from 1.5 at a depth of 2 ft. to 1.3 at 6 ft. and 1.08 at 10 ft.

Merging of Fields.—Ordinarily, when multiple outlets are used, the fields will merge a comparatively short distance from the rising columns. The merging of the individual fields may result in a thickening of the major field, an increased lateral velocity of spread, or a combination of both. The following data indicate that the increased lateral velocity predominates, although there is probably some thickening of the major field.

In calm water, with no currents, the major field can be considered as a number of individual fields each roughly corresponding in shape to a sector of the circular area of the major field with the column head near the apex. The condition obtaining in each individual field, under such a circumstance, was roughly reproduced by discharging a jet from a horizontal nozzle close to the surface, thereby imparting to the field a horizontal velocity in the direction in which the field would normally expand if its natural spread were laterally obstructed by other forces.

A tabulation of the results obtained from this experiment indicates that X_0 is about twice the X_0 value obtained when the field was permitted to expand equally in all directions.

In Equation (22), doubling the value of X_0 likewise doubles the value of x if $\left(\frac{S}{S_0}\right)$ remains constant. In the foregoing experiment the field was roughly circular and x represented the diameter. Under normal conditions of spread, without currents present, x will represent the radius. It follows therefore, that since the diameter of the field in the experiment was about twice the radius of the normal field the two are roughly equivalent.

This leads to the conclusion that even if the individual fields were still more distorted, approaching the shape of circular sectors, the assumption of equivalent areas would still hold. Results of investigations at the Hyperion Outfall indicate that to be the case.

The area of the major field, from such conclusions, will be n times the area of each individual field and its radius, or the distance the sewage will travel in calm water is obtained from the value of x in Equation (22), thus,

$$x = X_0 \left(\frac{S}{S_0}\right)^2 \sqrt{n} \ldots\ldots (29)$$

Due to the increased velocity of spread in the major field the time for Q to reach the limit, E, of the major field is the same as for $\frac{Q}{n}$ to reach the limit of its individual field.

Let the total flow through all the outlets be equal to,

$$Q' = n\, Q \ldots\ldots (30)$$

$$X_0' = X_0 \sqrt{n} \ldots\ldots (31)$$

$$F' = \frac{Q'\, S_0}{2\, \pi\, P \sqrt{X_0'}} \ldots\ldots (32)$$

Then, substituting Equations (30) and (31) into Equation (32):

$$F' = \frac{n\, Q\, S_0}{2\, \pi\, P \sqrt{X_0 \sqrt{n}}} = n^{\frac{3}{4}} \frac{Q\, S_0}{2\, \pi\, P \sqrt{X_0'}} = n^{\frac{3}{4}} F \ldots\ldots (33)$$

From Equation (17), the time required to reach the outer limit of the combined field is equal to:

$$t' = \frac{2}{3\, F'} (x')^{\frac{3}{2}} \ldots\ldots (34)$$

and substituting F' in Equation (33):

$$t' = \frac{2}{3\, n^{\frac{3}{4}}\, F} x^{\frac{3}{2}}\, n^{\frac{3}{4}} = \frac{2}{3\, F} x^{\frac{3}{2}} = t \ldots\ldots (35)$$

STRATIFICATION

It has been found that when water is undisturbed by wind or wave action, a difference of 2.5° Fahr.* in its temperature is sufficient to cause stratification. The equivalent of this difference is shown in Fig. 4, in which temperature of sea water is plotted against the specific gravity of the sewage for different values of S at 60° Fahr. From this, it may be seen that raising the temperature of sea water from 60° to 62.5° Fahr., reduces the specific gravity from 1.02540 to 1.02479, the latter being equivalent to the specific gravity of sewage at 60° Fahr., diluted to $S = 105$.

This indicates that if S is less than 105 the mixture will tend to come to the surface. Actual experiment shows, however, that mixtures with a much higher value of S will come to the surface, probably assisted by the inertia imparted by the initial velocity.

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* "Solving Sewage Problems," by Fuller and McClintock, p. 157.

After reaching the surface the lighter fluid spreads laterally over the sea water and its tendency to remain stratified is exceedingly persistent. Oceanographic evidences of stratification in clear sea water are common in places where only slight changes in specific gravity are apparent. Observations have disclosed that stratification temporarily exists where the heavier water forms the upper stratum, the layers thus being in a state of unstable equilibrium. They remain so until disturbed sufficiently to cause a re-adjustment to stability with the heavier water at its proper level.

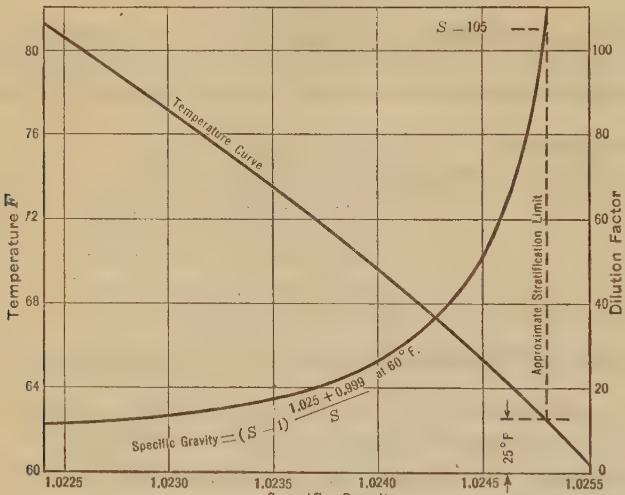

FIG. 4.

It is concluded, therefore, that the sewage field may not be expected to thicken before it has traveled such a distance from the column head as is needed to obtain a value in excess of 100 for S. The probable tendency will be for the sewage to remain distinctly stratified until a value of $S = 200$, or more, is reached; at which time (see Fig. 5), the specific gravities of the sea water and the dilute sewage are so nearly alike as to tend to merge, causing field thickening and consequent great dilution.

Wind and wave action or other disturbance will hasten the mingling of the thin sheet of sewage with the sea water and will have a pronounced effect on the breaking up of stratification; therefore, the most pronounced apparent fields will be observed during the periods of greatest calm.

In bacteriological surveys conducted about the Los Angeles City Outfall at Hyperion, a presumptive count of not to exceed 10 *B. Coli* per cu. cm. of sample taken is considered as marking the edge of the polluted area. Applying Equation (22) to the actual sewage field as found at Hyperion indicates that the value of S at the outer limit of the field is 212. Time as well as dilution may well be responsible for the bacterial destruction, and it is altogether probable that the sea water exercises a bactericidal effect on the sewage. It must be apparent that a presumptive count of 60 000 + *B. Coli* per cu. cm. of sewage cannot be reduced to 10 per cu. cm. of sewage and sea water by dilution alone in so short a distance from the column head as is usually found in bacteriological surveys.

It is undoubtedly true that, after the breaking up of stratification, the field thickens materially, reducing the lateral velocity, increasing the dilution, and accelerating the bactericidal possibilities of the sea water, and that, therefore, the place at which stratification breaks down marks approximately the outer limits of the field. This limit may be some distance beyond that place, however, and the actual value of S at its limits is much greater than Equation (22) would indicate at such location. For practical purposes, however, it is considered sufficient to neglect this thickening and apply Equation (22) on the assumption that S equals from 200 to 225.

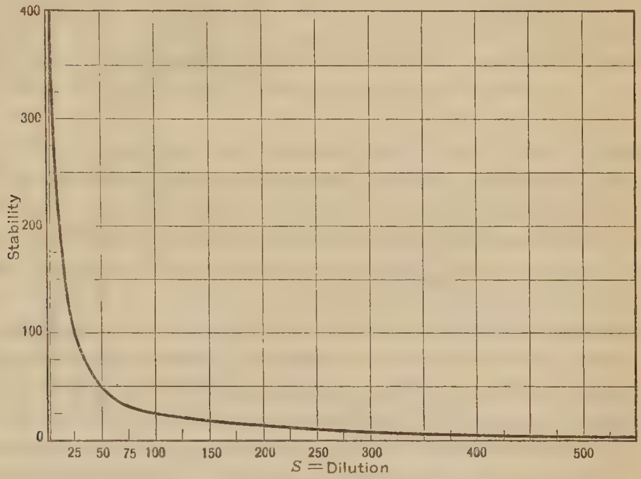

FIG. 5.—RELATION BETWEEN DILUTION AND STABILITY.

Following the assumption that the field stratification breaks down at a value for S of 200 +, care should be taken to provide sufficient depth of water at the onshore field limit to permit of material field thickening; that is, the depth of sea water at the shoreward limit of the field should be greater than the probable value for P.

CORROBORATION AND CORRECTION FROM RESULTS OBTAINED AT EXISTING OUTFALLS

Reference to Fig. 3 indicates that the upper limit of Curve A follows, within reasonable limits, the results obtained by measuring the actual value of S_0 obtained at four existing California outfall sewers.

At Long Beach, the ocean outfall consists of a 36-in. cast-iron pipe extending 850 ft. seaward along the ocean bed and discharging 8.5 ft. below mean sea level through a vertical riser.* The flow of sewage is continuous and during the series of experiments it varied from 6.7 to 10.5 cu. ft. per sec. The point of discharge is located in exceedingly calm water, and conditions over this outfall approach more nearly those under which the experiments at San Pedro were performed than at any other outfall site investigated.

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* Replaced in 1928 by 42-in. cast-iron outfall into 32 ft. of sea water, because of adjacent shore pollution.

At Santa Monica, the ocean outfall consists of a 24-in. pipe supported by the Santa Monica Pleasure Pier.* At the end of the pier it discharges vertically downward through a 20-in. pipe to a depth of 13.4 ft. below mean water level. During the period of observation, 7.5 cu. ft. per sec. was flowing, resulting in a velocity through the down pipe of 4 ft. per sec. and for the stage of the tide at that time, a value of L of 14.5 ft.

At Santa Barbara, sewage is discharged from a pumping station through a 42-in. reinforced concrete pipe laid in the ocean bed, the direction of discharge from the end of the outfall pipe being nearly horizontal and discharging the sewage 37 ft. below mean water level. The flow is intermittent; with one pump in operation 10.87 cu. ft. per sec. is discharged, while with two pumps the discharge is 15.2 cu. ft. per sec. During the period of observation one pump was able to empty the sump in about 4 min., and the interval between pumping was about 10 min.

At Hyperion, the City of Los Angeles has constructed a 7-ft. reinforced concrete outfall extending 5 300 ft. seaward from the shore line and terminating in a Y, with branches 320 ft. and 108 ft. long, respectively. This Y was constructed at the end of the outfall to divide the flow of sewage into two streams of equal quantity. Each leg terminates in an upward riser, so that sewage is discharged vertically upward.

On September 29, 1926, the City of Long Beach, in co-operation with the California State Board of Health, conducted a bacteriological survey to determine the extent of the polluted area surrounding the ocean outfall. The extent of the field was assumed by a limiting count of 10 *B. Coli* per cu. cm. of sea-water sample and was found to comprise 507 acres. The maximum sewage flow during the survey amounted to about 10.5 cu. ft. per sec.; or a polluted area of 48.3 acres per cu. ft. per sec. of sewage discharged. In this value of Q the writers differ with the city officials of Long Beach who state that the maximum was 15.3 cu. ft. per sec. Flow was computed by measuring head loss in passing a Republic flow meter disk, the recording apparatus being out of order at the time.

Equations (1) to (35) are not applicable to the determination of the size of a field under conditions existent at Long Beach. They would indicate a field far too large and one in which bacteriological destruction would occur long before the calculated outer limit of the field at $S = 200$ would be reached. Furthermore, they would indicate that a very thin layer of mixed water would have to maintain itself as an independent stratum for a period extending into days. Influenced by the tidal currents into and out of the Long Beach Inner Harbor Entrance, sewage from the Long Beach Outfall would reach the outer limits of the actual field, as found, in 4 to 6 hours.

At the same time that the City of Long Beach was conducting the bacteriological survey the writers made a separate survey of dilutions and surface velocities within a radius of 200 ft. of the outlet. The sewage is discharged between the two breakwaters which protect the Inner Harbor Entrance and about 200 ft. east of the channel into the Inner Harbor; it is, therefore, subject to an eddy current. Measurements of the dilution were made in three directions from the column head, from which it was found that the value of S was 2.8 $x^{0.31}$, 0.58 $x^{0.32}$, and 2.5 $x^{0.32}$ in the respective lines of travel. The differences were due to the effect of the eddy current. Substitute in Equation (29), the known value of $S_0 = 2$, a mean value of $S = 1.2\ x^{0.5}$ and $n = 1$; then, x cancels and $X_0 = 2.8$. By applying the same values in Equation (23), X_0 is found to be equal to 2.6 which agrees with 2.8 within the limits of error allowed for the observation.

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* To be replaced in 1929 by connection to Los Angeles City System because of adjacent shore pollution.

A rough check on the thickness of the field was obtained from the velocity at which the flow of sewage left the column head. This was made by observing the time required for chips of wood to float past fixed points in one direction from the outlet. The observations follow the formula, $t = 0.08\ x^{1.58}$, which, by differentiation, results in the equation:

$$V = \frac{7.9}{x^{0.58}} \qquad \text{(36)}$$

The field thickness, P, may be obtained by first combining Equations (19) and (36) and then substituting the known values, $Q = 14.3$ and $S = 1.2\ x^{0.50}$. For $x = 15$ ft., the value of P is found to be 0.42 ft., and for x 200 ft., $P = 0.36$ ft. Assuming the mean velocity of the sewage stratum to be 0.8 of the surface velocity, as in float measurements, the field thickness at 15 ft. becomes 0.53 ft. as compared to 0.75 ft. obtained from Equation (16). This observation of field thickness is noted for the purpose of illustrating the shallow depth of the sewage field.

Twice each year the City of Los Angeles conducts a bacteriological survey in the vicinity of its ocean outfall at Hyperion, which is intended to determine the area of ocean pollution. Owing to the fact that shortly after the construction of the outfall numerous leaks of considerable magnitude appeared at the joints between concrete pipe sections and that the leaks were not repaired until about April, 1927, there are only five of the surveys which interpret conditions as they should exist with the sewage discharge confined to definite, known outlets at or near the seaward end.

TABLE 3.

Date.	Q, in cubic feet per second.	Observed area of field, in acres.	Acres per cubic foot per second.	Number of outlets.	Depth, in feet.	S_0.	Computed area, in acres.
January 11, 1925..	120	720	6.0	1	57	10.5	710
May 17, 1927.......	200	555	2.7	3	56	14	524
October 4, 1927....	170	986	5.8	2	55	11.5	(986)
February 29, 1928..	197	834	4.2	4	52	15	1 090
May 23, 1928.......	205	1 075	5.2	4	52	14.5	1 170

In Table 3 are the results of these five surveys, together with the theoretical results supplied by application of Equations (1) to (35) to conditions as they existed on the dates of the survey.

The comparison of the field spread in each case (see Table 3) is made on the assumption that the value of X_0 is alike in each case, that it is correct within the reasonable limits of error, and that the outer boundary of each field is fixed by a value of $S = 212$. This was the value of S at the field boundary as determined by Equations (1) to (35) for the survey conducted October 4, 1927.

The advantage of initial dilution is well indicated by a comparison of these three fields with each other and with those at Long Beach. Between February or March, 1925, and April, 1927, the line leaked badly and could readily be compared with a line having a large number of small outlets. Each bacteriological survey conducted during this period indicated a smaller area of pollution per second-foot of discharge than is shown by any one of the five surveys given in Table 3.

At Deer Island, in Boston, Mass., Harbor, about 1920* the sewage from a population of 700 000 was discharged horizontally in 30 ft. of water through fourteen rectangular outlets, 18 by 30 in. in size, distributed through a distance of 120 ft. Observations showed that the value of S_0 was 21 and that 300 ft. away from the outfall pipe, sewage could not be visually detected. Several hundred feet away, chemical analysis showed about 100% sea water.

Assuming a rate of 100 gal. per capita daily, from the tributary population, the maximum total flow through the Deer Island Outfall would amount to 108 cu. ft. per sec., or 7.7 cu. ft. per sec., flowing through each outlet. The corresponding length of path should be 36 ft. and, assuming that the outlets are arranged in pairs discharging in opposite directions so that rising columns do not interfere, $S_0 = 19$ theoretically. For $x = 300$, $S = 110$, and for $S = 212$, $x = 2\,100$ ft.; the field area is 317 acres, or 2.94 acres per sec-ft., corresponding to a field limited by the same conditions of final dilution as the observed field at Hyperion. If the observed value ($S_0 = 21$) of the initial dilution is used, the field limit, where $S = 212$, is found to be at $x = 1\,715$ ft. and the area is 212 acres. As far as can be determined from data available, Equations (1) to (35) indicate a field on the side of safety.

Probable Error

The probable error in the value of S_0, as determined from several of the vertical observations at constant depths, amounts to approximately 4 per cent. If a second variable is introduced, which in this case is the length of path, it would cause this to be multiplied by $\sqrt{2}$, making the resulting probable error about 6 per cent.

Since x, the distance to the outer boundary of the field, is inversely proportional to the square of the initial dilution, it is subject to an error of 12%, and the area of the field to an error of 25 per cent. Therefore, while so much depends on the value of S_0, it is a useless refinement to compute its value, and it can be found from Fig. 7 within the limits of error.

All the experiments at San Pedro Harbor were performed in very quiet water, and at Long Beach, the water at the outfall discharge is almost equally as smooth. At Santa Barbara there are small swells and the outfalls at Santa Monica and Hyperion are in the open ocean.

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* Rept. of Special Sewage Disposal Comm., Los Angeles, Calif., August 10, 1921.

It has been considered axiomatic that, other conditions being equal, discharge into rough, disturbed water would result in greater dilution than discharge into calm water. The experiments at San Pedro indicated this clearly, in that relatively larger values of S_0 were obtained with the use of $\frac{1}{4}$-in. and $\frac{3}{8}$-in. nozzles, than with the large sizes. The explanation for this is that, even in the more sheltered area, there were always present, currents of sufficient magnitude to effect results from these small streams of discharge, and these had little effect on the larger ones.

Extending this reasoning to nozzles of a size likely to be used in outfall sewer construction, it may be seen that on a calm day little, if any, increase should be found over the theoretical value in the dilution. When pollution is fixed by its effect on neighboring bathing beaches, calm weather conditions should be especially taken as a criterion for obvious reasons.

Application of Equations (1) to (35)

If the horizontal jets are to be used, the length, L, may be found from Equations (8) and (9) by introducing the value of x given in Equation (7); or, as circumstances warrant, may be taken from Fig. 6. For vertical jets, $L = y$.

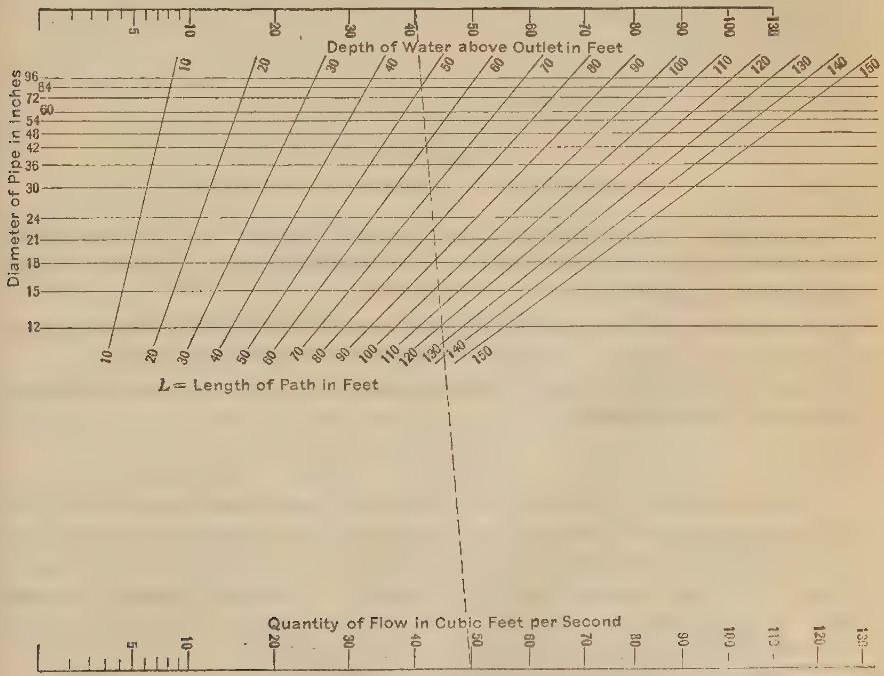

Fig. 6.—Length of Path from Outlet to Water Surface for Horizontal Outlets.

The initial dilution, S_0 may be found from Equation (10), or Fig. 7. (It should be noted that in Equation (10), Q is in gallons per minute.) The total quantity of water coming to the surface is $Q\ S_0$, of which, $Q\ (S - 1)$ is salt water.

The distance to which the sewage will travel away from the outlet, if there are no currents present, is found from Equations (23) and (29). This distance may also be taken from Fig. 8 with $S = 225$. For any other value of S multiply x by $\left(\frac{S}{225}\right)^2$. In some cases, x and t may exceed the limits of Fig. 8, but then the time element will be so great that other factors will enter and make the formulas inapplicable.

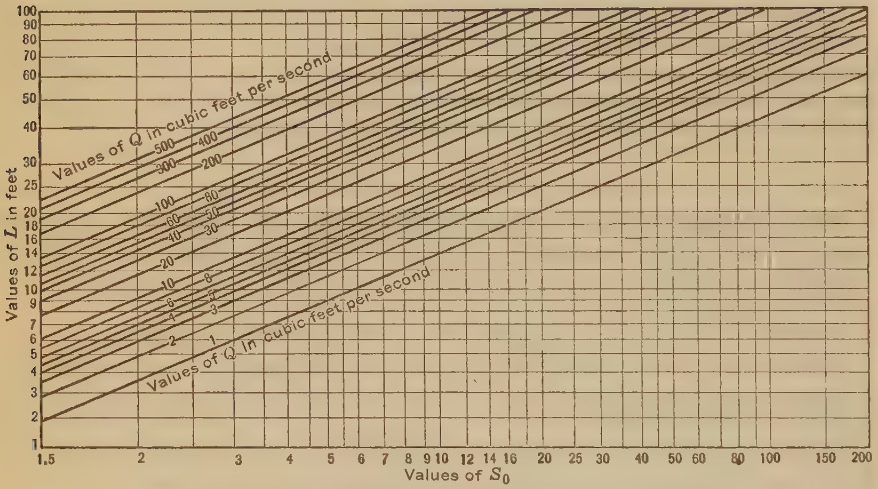

Fig. 7.—Initial Dilution of Sewage in Salt Water.

The distance the sewage will travel with the current may be determined from Equation (28), in which, x and t are found from Fig. 8 and U is the velocity of the ocean current.

If it is desired to determine the distance that the sewage will travel against an ocean current, use may be made of Equations (16), (21), and (27).

To compute the time required by the sewage to reach the outer boundary Equation (17) is applicable.

Examples

As an illustration of the application of these equations assume that 100 cu. ft. per sec. is to be discharged at a depth of 40 ft. below the ocean surface, at a location where a directly onshore current of 30 ft. per min. is considered.

Case 1.—First, consider that the discharge is vertical, through a single outlet, 72 in. in diameter; then, $Q = 100$; $L = y = 40$ ft.; $D = 72$; $n = 1$; $U = 1\,800$ ft. per hour, or 30 ft. per min.; and $S = 225$.

From Fig. 8 the radius of the field at $S = 225$ is more than 10 000 ft., so it will be necessary to compute x and t. Ordinarily, this is not worth while as other factors will tend to limit the field, but the computation is carried out as an illustration.

From Fig. 7, $S_0 = 5.8$. By Equations (23), (29), (16), (21), and (17): $X_0 = 8.0$; $x = 12\,000$ ft.; $P = 3.3$ ft.; $F = 9.9$; and $t = 24.5$ hours, respectively. Hence, $U\,t = 44\,500$ ft., and, from Equation (28), $e = 56\,500$ ft., which is the theoretical distance.

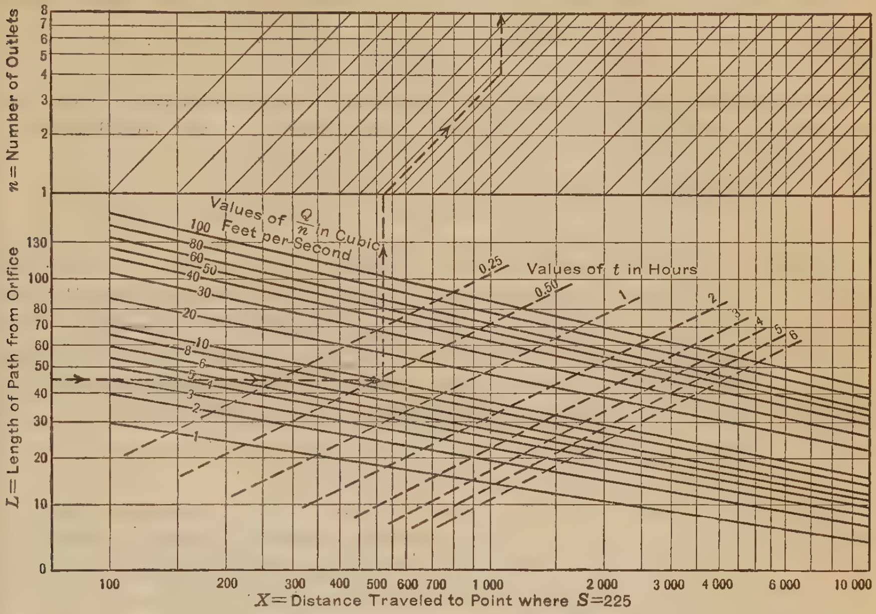

FIG. 8.—MEAN RADIUS OF SEWAGE FIELD.

By Equation (27), $e = 131$ ft., which is the distance the sewage will flow against the current. The shape of the sewage field will be roughly an ellipse with the major axis equal to the sum of the distances with and against the current, or 56 630 ft., and the minor axis, $2\,x = 24\,000$ ft. The area will be,

$$A = \frac{\pi}{4}\left(\frac{56\,630 \times 24\,000}{43\,560}\right) = 24\,500 \text{ acres, or 245 acres per cu. ft. per sec.}$$

It is very probable in the case under consideration, that the thin stratum will not exist as such for long and that time will limit the field. Assuming 5 hours as the field limit, then, from Equations (17) and (28), respectively, $x = 4\,150$ ft. and $e = 13\,150$ ft. From this value the area is found equal to 1 990 acres, or 19.9 acres per cu. ft. per sec.

Case 2.—Assuming the use of two vertical 54-in. discharge nozzles, let $Q = 100$ cu. ft. per sec.; $n = 2$; $\frac{Q}{n} = 50$ cu. ft. per sec.; $y = L = 40$ ft.; $D = 54$ in.; $U = 1\,800$ ft. per hour, or 30 ft. per min.; and $S = 225$.

To prevent interference between the rising columns below the surface, the two outlets should be at least $\frac{40}{3} + 4.5 = 17.8$ ft. apart. The value of S_0, in this case becomes 8.4. This factor is not needed in the computation, but is mentioned to emphasize the effect of S_0 on the size of the sewage field. By Fig. 8, $x = 7\,600$ ft. The time being in excess of 6 hours, is not shown in Fig. 8 because the size of the field will probably be limited by other factors.

By means of Equations (23), (16), (21), and (17): $X_0 = 7.25$ ft., and $X_0 \sqrt{n} = 10.25$ ft.; $P = 3.3$ ft.: $F = 12.6$; and $t = 9.55$ hours, respectively. Hence, $U\,t = 17\,200$ ft.

By Equation (28), $e = 24\,800$ ft., the distance sewage will travel with the current and by Equation (27) $e = 212$, the distance it will travel against the current. As before, the area of the sewage field will be,

$$A = \frac{\pi}{4}\left(\frac{25\,012 \times 15\,200}{43\,560}\right) = 6\,900 \text{ acres, or 69 acres per cu. ft. per sec.}$$

Limiting this field by a 5-hour time period as in Case 1, then, by Equation (17), $x = 4\,850$ ft., and by Equation (28), $e = 13\,850$. The field area then is 2 450 acres, or 24.5 acres per cu. ft. per sec. It is probable that the combination of time and greater dilution in Case 2 will tend to break down stratification earlier than in Case 1; nevertheless, it is considered advisable to limit such a field, as is under consideration, to not less than 5 hours.

TABLE 4.

		Case 3.	Case 4.	Case 5.	Case 6.
Given:	n	1	2	4	6
	Q, in cubic feet per second	100	100	100	100
	$\frac{Q}{n}$, in cubic feet per second	100	50	25	16.7
	y, in feet	40	40	40	40
	D, in inches	72	54	30	27
	U, in feet per hour	1 800	1 800	1 800	1 800
By Fig. 6:	L, in feet	58	55	57	55
By Fig. 7:	S_0, in feet	11.5	15.5	26	30
By Fig. 8:	x, in feet	3 600	2 700	1 360	1 100
By Fig. 8:	t, in hours	3.25	1.75	0.47	0.33
	$U\,t$, in feet	5 850	3 150	840	600
	$e = x + U\,t$, in feet	9 450	5 850	2 200	1 700
	X, in feet	10.25	9.1	8.38	8.1
	$X \sqrt{n}$, in feet	10.25	12.9	16.75	19.8
	P, in feet	4.8	4.6	4.75	4.6
	F, in feet	11.9	15.45	21.3	23.3
	$\frac{1}{3}\left(\frac{F}{U}\right)^2$, in feet	203	320	605	725
	A, in acres	1 250	601	137	88
	$\frac{A}{Q}$, in acres	12.50	6	1.4	0.9

If horizontal outlets are used, the computation is the same as for vertical outlets, except that it is first necessary to determine the length, L, of the rising column from Fig. 6, or by means of Equations (8) or (9). Equation (9) applies only to shallow depths with high velocities at the outlet, where the horizontal distance from the outlet to the point where the column reaches the surface is equal to or greater than three times the depth. For most cases Fig. 6 will suffice instead of Equation (8). Four other cases are given in Table 4.

DISCUSSION

GEORGE W. FULLER,* M. AM. SOC. C. E. (by letter).—The authors have presented interesting observations with equations corresponding to a variety of assumptions as to the travel of sewage in sea water. Of particular interest is the application of equations to conditions at existing outfalls. It would be advantageous to extend such applications to other outfalls.

From the standpoint of pollution of bathing beaches or shellfish layings it is probable that sewage fields are moved for greater distances at times of heavy wind storms than is generally realized. Public health aspects make it particularly desirable to diffuse the sewage as extensively as practicable and thus eliminate the likelihood of any sewage field at all. Such a result is related to the use of multiple outlets which have substantial submergence, and to the release of sewage under conditions by which its velocity of discharge is quickly and almost completely checked, as well as to its release in numerous small streams whereby quick and thorough dispersion is promoted.

The writer's observations indicate that there is no more adequate design for the discharge of sewage into brackish water than the arrangement provided for the Passaic Valley trunk sewer, designed to serve a population of 1 600 000 and discharging sewage into Upper New York Bay near Robbins Reef Light. The 150 nozzles or diffusers, located about 50 ft. below mean low water and distributed over an area of about 3 acres, release the sewage with a swirling outward motion so that it leaves the periphery of the nozzle in a thin ribbon. Dispersion of the sewage is practically complete.

This paper is also of interest to engineers who have to do with the disposal of sewage in the Great Lakes or other large bodies of fresh water where there is opportunity for the discharge to pollute bathing beaches or water-works intakes. There are different factors, of course, in such a situation as compared with outfalls in sea water. Stratification conditions are different and the effect of tides is absent, which in estuaries like New York Bay never allow the water at various depths to come to a complete standstill at the same time. Furthermore, in the Great Lakes, the effect of wind at times may be such as to cause the movement of sewage fields for relatively great distances, thus showing the need of effective dispersion.

KENNETH ALLEN,† M. AM. SOC. C. E. (by letter).—The phenomenon of the course and diffusion of sewage discharged from submerged outlets is one that has rarely received close study, although the general results have been long recognized. The authors have attacked the problem in such a way that, under given conditions, the form of the issuing jet and its spread over the surface of sea water may be approximately predicted. This is believed to be an important advance over previous investigations along these lines.

The experiments and conclusions drawn therefrom are, however, limited to sea water, and in the application of the deduced formulas it is possible that the inexperienced engineer might not appreciate the important influence of salinity and the direction, form, and diffusion of the sewage stream. In inland cities the phenomena accompanying the discharge of sewage from submerged outlets in lakes and rivers will differ radically from those to be observed in salt water.

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The effect of salinity was brought out very clearly in experiments made by the writer for the Metropolitan Sewerage Commission of New York in 1909. Owing to the configuration of the harbor, the flow of upland water brought down by the Hudson River and the action of the tides, the percentage of sea water was found to average 74.8 at West Bank Light in the Lower Bay and 33.7 in the Hudson River off Fort Washington Point. The extreme readings noted at those points during the year were 93.2% at West Bank Light on November 5, and 0% at Fort Washington Point on April 8. Conditions varied, therefore, from those of fresh water to those of nearly pure sea water.

Several series of experiments were made to learn more of the action of sewage discharged into waters of different specific gravities.

Experiments on Flotation.—Croquet balls were weighted until they balanced in water of a specific gravity of 1.000, and then released in water of known specific gravity at a given depth.* The time required for the balls to rise to the surface was noted with a stop-watch. (See Table 5.)

The results, while not entirely consistent, give a fair idea of upward velocities of smooth solids released in waters of different density that will not be reached by a diffusing liquid. Bottles, released in a similar way, rose even more rapidly, on account of their smoother surfaces.

Laboratory Experiments in Diffusion.—A total of thirty-one experiments was made to observe the action of a dyed sewage or water when discharged from a pipette, or small tube, in waters of different salinity. Alcoholic solutions of eosin and fluorescein and aqueous solutions of bluing, permanganate of potash, and uranine were tried. The dye, especially the bluing, was found to increase the specific gravity slightly, and it was feared that alcohol might affect diffusion. Permanganate of potash was likely to be decolorized by organic matter. It was finally decided to use uranine ($C_{20}H_{10}O_5Na_2$) which, in a strong solution, is a deep red, but becomes a brilliant green on dilution, readily detected in a Nessler tube in a 1 : 20 000 000 dilution. A dye called "Special Scarlet" was used later and was found to be a very serviceable and less expensive substitute.

These experiments indicated clearly the effect of specific gravity and the way in which a drop or a jet diffused. Some of the forms observed were beautiful as well as interesting: Waves of floc; rings like smoke rings; funnels; and cones and wavy threadlike lines, rising or falling, and diffusing rapidly or slowly. The diffusion of 2 cu. cm. of a 1% solution of uranine added gently to the surface of 200 cu. cm. of waters of varying salinity in beakers and jars began in a few minutes, was complete in an hour with tap water, and took 48 hours in a mixture containing 25% of sea water, while it had not occurred with a 75% mixture of sea water in a week's time. (See Table 6.) Salinity, therefore, was shown to have a marked effect on diffusion.

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* The salinity of sea water varies in different parts of the ocean, but for this region chlorine of 18 000 parts per million, corresponding to a specific gravity of 1.025, was assumed as constituting 100 per cent.

Experiments in Diffusion in the New York City Aquarium.—These experiments, twelve in number, were made to confirm the results obtained in the laboratory before conducting others on a larger scale in the open water. The tank was 4 ft. high, 4 ft. wide at the top, narrowing to about 3 ft. at the bottom, and 8 ft. long, with an overflow a few inches below the top. The dye and water were fed from a funnel and by a tube to a nozzle, the depth and direction of which were adjustable. Nozzles of ¼ in., ⅜ in., and ½ in. were used.

TABLE 5.—EXPERIMENTS TO DETERMINE UPWARD VELOCITY OF SMOOTH SOLIDS.

Experiment No.	Number of tests.	Specific gravity of water.	Vertical distance traveled, in feet.	Average velocity, in feet per second.
1	13	1.0215	1.47–1.51	0.500
2	10	1.016	1.70	0.455
3	15	1.007	1.62	0.252
4	15	1.009	1.62	0.301
5	4	1.0195	10.0	0.655
6	10	1.0215	10.0	0.49

Without going into the details of these experiments, some of the results were as follows:

(*a*) With a horizontal jet a considerable current may be induced at a slight depth without apparent effect on the surface itself.

(*b*) With a ¼-in. horizontal jet of fresh water in fresh water, under a head of 1.0 ft., the axis of the jet remained horizontal, enlarging in diameter to 3 in. at a distance of 1.25 ft. from the nozzle and to 6 in. at a distance of 3.0 ft. from the nozzle, and then diffusing slowly. After substituting a jet of sewage, the diameter 3.0 ft. from the nozzle was 12 in.

(*c*) In the case of a ¼-in. upward jet of fresh water at a depth of 3.0 ft. in fresh water under a head of 2.5 ft., the jet formed a concave cone 5 in. in diameter 1.0 ft. from the nozzle and spread in a thin layer at the surface. By the end of 5 min. diffusion was quite uniform to a depth of 1.5 ft.

(*d*) With a ½-in horizontal jet of sewage in fresh water near the bottom of the tank, 3.4 ft. below the surface and under a head of 0.15 ft., there was a gradual descent of the jet to the tank bottom which it covered with a blanket about 2 in. thick, with very little diffusion, in 3 min. The temperature of the sewage was 62° Fahr., with a specific gravity of 1.001. The temperature of the water was 61° Fahr., and its specific gravity was 1.000.

(*e*) With a ½-in. jet (nearly horizontal) of sewage in harbor water, at a depth of 3.3 ft. and under a head of 0.15 ft., a temperature of sewage, 62° Fahr., specific gravity, 1.0015, and a temperature of water, 58° Fahr., the effluent rose promptly in an upward curving cone, reaching the surface in 1 min., where it spread out. At the end of 3 min. the supply of dye was cut off and by the end of 5 min. there remained but a thin green layer at the surface.

(*f*) For a ¼-in. horizontal jet of sewage in harbor water, as in Result (*e*), under a head of 1.0 ft., the diameter in a plane 1.0 ft. above the nozzle was 8 in. (see Fig. 1). The horizontal component of the jet did not appear to affect the time taken to reach the surface.

(*g*) With a ¼-in. upward jet of sewage in harbor water, at a depth of 2.5 ft., and under a head of 1.0 ft., the sewage rose promptly to the surface with little diffusion and then spread in a thin layer. The diameter 2 ft. above the nozzle was 6 in.

(*h*) At a ¼-in. downward jet of sewage in harbor water, under a head of 1.0 ft., the jet rose as a column of greater diameter and with more rapid diffusion than in Result (*g*) and spread at the surface. Diffusion was increased materially by a slight agitation of the water.

TABLE 6.—DIFFUSION IN WATERS OF DIFFERENT SALINITY.*

Liquid added.		Top water.	Sewage.	Sewage.
Dilution of uranine solution..........		1 : 100	1 : 1 000	1 : 10 000
Specific gravity of liquid added......		1.000	1.001½	1.001½
Percentage of sea water.	**Percentage of tap water.**	**Diffusion apparently took place between**		
100	0	2 and 24 hours.	6 and 48 hours.	....
75	25	2 and 24 hours.	6 and 48 hours.	Not complete at end of week.
50	50	2 and 24 hours.	6 and 48 hours.	56 hours.
25	75	5 min. to 2 hours.	6 and 24 hours.	48 hours.
0	100	At once.	At once.	Few minutes and 1 hour.

* Experiments by Dr. Payne B. Parsons, Bacteriologist, Metropolitan Sewerage Comm. of New York.

The results of these experiments are generally confirmed by the work of the authors and by a similar apparatus, but their analysis has been carried to a point where the path of a jet of fresh water in sea water may be more closely predicted.

Experiments on Diffusion in New York Harbor.—Nine experiments were made by placing a dye in the surface water of the harbor, or at sewer outlets, to observe the rate at which sewage might be expected to become diffused and dispersed under different conditions. These were followed by twenty-one experiments to observe the result of discharging dyed water at various depths and locations in the harbor.

In three experiments at the outlet proposed at that time for the Bronx Valley Sewer, in the Hudson River, off Mount St. Vincent, dyed water was pumped at rates of 17 to 25 gal. per min., to depths varying from 5 to 25 ft. through a 1¼-in. hose. The velocity of the river water varied from 0 to 3 ft. per sec., and its specific gravity varied from 1.010 to 1.013. The detection of dye at the surface was uncertain at slack water from depths greater than 16 ft., or in a current of 2½ ft. per sec. from depths greater than 9 ft.

Nine similar experiments were made at various points in New York Bay, Gravesend Bay, and the Upper East River. These indicated that:

(*a*) With a difference in specific gravity of 0.004, dyed water will rise in quiet harbor water from depths of 10 to 26 ft. at a mean rate of 0.10 to 0.14 ft. per sec., and be visible at the surface.

(*b*) With a difference in specific gravity of 0.004½, it will rise from a depth of 18 ft., at a mean rate of 0.17 ft. per sec. and be visible at the surface.

(*c*) With a difference in specific gravity of 0.016, it will rise from a depth of 27 ft. in a current of 1½ ft. per sec., at a mean rate of about 0.10 ft. per sec., and be visible at the surface.

(*d*) In moderate currents dyed water may be expected to be visible at the surface if discharged at depths of 15 to 20 ft., but that the rate of ascent is variable. The limit from which it can be discovered under the most favorable conditions appears to be about 40 ft. In sea water (specific gravity, 1.021½), with a current of 0.3 ft. per sec., diffusion is very gradual at first, but proceeds more rapidly later.

Two similar experiments, but on a larger scale, were made in the Upper Bay, near the site of the Passaic Valley Sewer outlet off Robbins Reef, which has since been built.

In the first experiment, 16 500 gal. of a 1:2 750 solution of uranine in tap water was discharged 40 ft. below the surface through a 2½-in. hose, at a rate of 205 gal. per min., giving a velocity of 14 ft. per sec. The specific gravity of the harbor water varied from 1.0115 to 1.0135.

In the second experiment, 56 000 gal. of tap-water containing 290 lb. of uranine, forming a solution with a specific gravity of 1.003, was pumped from a pontoon through an 8-in. pipe, at a rate of about 2 320 gal. per min., or with a velocity of 14.8 ft. per sec., to a depth of approximately 40 ft., the pontoon and discharge pipe rising as the experiment progressed. The specific gravity of the harbor water was 1.015, and the velocity of its current, as observed by submerged floats, averaged: 0.45 ft. per sec. at a depth of 3 ft.; 1.03 ft. per sec. at a depth of 20 ft.; and 2.85 ft. per sec. at a depth of 50 ft.; from which the mean velocity at the point of discharge was taken as 2.2 ft. per sec.

In spite of these large volumes no evidence of the discharge was observed in either case, although it had been detected once before in one of the smaller experiments at this point and at the same depth. The inference was that diffusion occurred rapidly due to subsurface currents at the time of the larger experiment, and that the rise of sewage to the surface probably depends more on the velocity of such currents than on the volume of sewage discharged.

It is clear that salinity of a body of water receiving sewage is an important factor in its diffusion, as it also is in its oxidation, its effect on precipitation, its decomposition, its general appearance, and its accompanying odors. The sewage of the City of Washington, D. C., discharged in 30 ft. of fresh water in the Potomac River and that of Hamburg, Germany, discharged at a depth of 30 ft. in the Elbe, is difficult to discern at the surface, while that from the Boston and New York outlets forms a field on the surface that is often clearly visible, well defined, and extensive. It was noted in the New York experiments with dyes that the edge of the field on the windward side was most clearly defined, while the more rapid diffusion took place on the leeward side.

From what has been said it is evident that diffusion is promoted by locating inlets in deep water of low salinity, but having a high velocity of flow. Discharge should be horizontal and from multiple outlets. The submerged outlets from Deer Island in Boston Harbor, with fourteen horizontal orifices, and the one hundred and fifty outlets of the Passaic Valley Sewer, in which the upward-flowing stream is dissipated by spiral flanges, are excellent examples of effective diffusion and dilution of an effluent in salt water.

TABLE 7.—Effect of Depth on Percentages of Sea Water and Dissolved Oxygen near Sewer Outlets in Boston Harbor, Massachusetts.

Date, 1911.	Distance from outlet, in feet.	Outlet.	Depth below the surface, in feet.	Percentage of sea water.	Dissolved oxygen percentage of saturation.
August 29........	10	Deer Island	1	68	53
			7	89	88
August 30........	90	Deer Island	Near surface to 1	80	80
			2	88	95
			3–60	94	100
September 6........	1 000	Moon Island	Near surface to 1	82	73
			1	82	75
			2	86	90
			3	88	95
			5	90	98
			10	92	99
			15	94	100
September 7	500	Moon Island	Near surface to 1	80	68
			2	86	78
			3	88	88
			5	88	94
			10	90	97
			20	92	100
September 11........	1 000	Moon Island	Near surface to 1	78	68
			2	82	76
			3	86	87
			5	88	92
			10	92	97
			15	94	100
September 11........	4 600	Moon Island	Near surface to 1	80	72
			2	84	85
			3	88	92
			5	90	97
			10–20	94	100
September 14........	2 000	Moon Island	Near surface to 1	82	68
			2	86	73
			3	88	85
			5	90	92
			10	94	97
			15	96	100
September 14........	2 500	Moon Island	Near surface to 1	84	72
			2	86	80
			3	88	86
			5	90	97
			10	94	100
			15	96	100
September 18........	5 280	Moon Island	Near surface to 1	86	75
			2	88	82
			3	90	89
			5	90	97
			10	94	100
September 18........	1 000	Moon Island	Near surface to 1	82	68
			2	86	74
			3	88	85
			5	90	92
			10	92	97
			15	92	98

As the visibility of sewage fields in calm weather is largely due to the oily surface film known as "sleek", it is obvious that for the sake of appearance, oily wastes should be kept out of effluents so far as practicable: First, by control and treatment at the source of industrial wastes of this nature; second, by providing municipal collection of crank-case wastes from garages; and, third, by installing skimming tanks on the line of outfall sewers. Other points that might be emphasized are the very great effect of turbulence of current on diffusion, and the variable effect of winds on the shape and extent of the sewage field.

Table 7* shows how the sewage discharged at the Deer Island and Moon Island outlets in Boston Harbor concentrates near the surface, as indicated by the increasing salinity or percentage of sea water and saturation with dissolved oxygen with increasing depth.

R. F. Goudey,† Assoc. M. Am. Soc. C. E. (by letter).—The authors are to be congratulated on their presentation of basic data derived from unique and ingenious experiments, which now definitely removes "judgment" as the sole factor to be considered in choosing sizes and lengths of ocean outfall sewers. Although the paper abounds in involved mathematical discussion, which most sanitary engineers shun, an impartial and critical analysis indicates the logic behind the important and significant conclusions which the authors have drawn from their experimental data.

Disposal of sewage and the degree of prior treatment required in connection with ocean disposal need particular study in Southern California, because of general unfavorable conditions, as compared with the Atlantic Coast. Instead of a Gulf Stream, with good currents relatively close to shore, as prevails along the Atlantic, California's main current, the Japan Current, runs more than 1 000 miles from shore. The littoral currents are wind-induced, low in velocity, variable in direction, and undependable for sewage disposal. The wind direction, likewise, is variable, due to changing from land to ocean breezes daily. For instance, during the night the land breezes prevail from north to east, and, finally, work around to the southeast in the early morning. With the overcoming of the land breezes by the ocean winds during the forenoon, the prevailing and strongest winds of the day come from the southwest. Toward evening, they veer toward the north, when the land breezes dominate. Except during stormy periods, when the winds are either from the north or southeast, this continual routine of "boxing the compass" accounts for the lack of uniformity in dependable shore currents. For these reasons, the authors are correct in using calm periods as critical.

A rather high standard of ocean disposal has naturally resulted in Southern California, as its coast line alternates with wide sandy beaches, which are ideal for bathing, and rocky bluffs, which are fast being utilized for picturesque home sites. Experiments to correlate bio-chemical oxygen demand, dissolved oxygen, and chlorides, with sewage fields, failed to establish a sensitive relationship or a law. Experience indicates that the sewage field loses its identity as badly discolored water at a *B. Coli* index of 10 per cu. cm., and this is the standard now used by sanitary engineers in California. It is believed to be a reasonably fair basis as compared to the accepted drinking water standard, considering relative quantities of water imbibed for drinking purposes, compared with that imbibed during bathing, nor does the figure differ from the *B. Coli* content common to natural streams having no unfavorable hygienic record. Adoption of less stringent standards, say, 50 *B. Coli* per cu. cm., which might safely be allowed during "slips", is not of much economic importance, because of the closeness of the two zones. Further requirements of ocean disposal dictate that there should be no odor in the vicinity of the outlet, nor sewage litter stranded alongshore.

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* Determinations by Dr. Payne B. Parsons, abstracted from the Report of the Metropolitan Sewerage Comm. of New York, 1912, Chapter III.

† San. Engr., Bureau of Water and Power, Los Angeles, Calif.

In the earlier outfalls in Southern California, the length from shore was set arbitrarily, even in the case of large outfall sewers, such as that for Los Angeles. The lengths of the more recent outfall sewers have been gauged from the behavior of *B. Coli* fields of pollution determined from pre-existing outfalls. The outlets are generally made to discharge from up-turned ells lying on the ocean floor, in order to avoid their becoming clogged with shifting sand, and to prevent the transmission of water hammer.

Table 8 gives a few examples of long outfalls in California that have a vertical discharge.

The outfall is rather long, compared with those one would design by use of the formulas given by the authors.

For small communities, the ease of using pure and protected wrought-iron pipe as large as 18 in. in external diameter, has not caused any particular hardship, in achieving the lengths of outfalls actually used. Although the life of such outfalls may not be more than twenty-five years, for growing cities, they represent an easy way to obtain a serviceable outfall for a fairly long term of years.

TABLE 8.—LONG OUTFALLS WITH VERTICAL DISCHARGE.

City.	Population, present.	Treatment.	Outfall.		
			Diameter, in inches.	Length, in feet.	Depth, in feet.
Los Angeles	1 250 000	Screening	84	5 400	63
Orange County	60 000	Screening	42	2 700	40
Santa Monica*	50 000	Screens	24	2 000	25
Santa Barbara	40 000	Screening	42	3 694	40
Ventura	20 000	Screening	17	2 000	30
Venice	20 000	Screening	14	1 360	25
Santa Cruz	16 000	Screening	2–15	2 000	37
Watsonville	10 000	Screening	16	1 500	20
Oceanside	5 000	Septic tank	11	2 000	33
Mission Beach	5 000	Septic tank	14	1 200	22
Pismo	3 000	Septic tank	11	1 700	17
Pacific Beach	3 000	Septic tank	14	1 500	15
Laguna Beach	2 000	Septic tank	10	600	15

* Downward discharge.

In larger cities, the cost per foot has been as much as $150, and, in some proposed projects, the cost has been variously estimated up to $500 per ft. Obviously, any factors that may shorten the outfall, or establish its performance in advance, deserve analysis. Investigations to perfect ocean disposal are quite as important as the efforts of inland communities to refine their high-grade treatment processes. Along recreational and residential shores, the ocean should not be regarded as a crude dumping ground for sewage.

There are certain factors not stressed in this paper which probably have quite a bearing on the performance of ocean disposal outlets. From the time sewage leaves the outlet until it reaches the surface, dilution and diffusion appear to yield an extremely rapid and extensive purification. The greater the oxygen content of the water, and, conversely, the less the oxygen demand of the sewage, the less will be the area of surface pollution. The quantity of dissolved oxygen in sea water varies according to the temperature of the water and the amount of vegetable life in it, which at certain times even causes supersaturation with oxygen of 5 or 10 per cent. The oxygen demand value of the sewage varies with its age and treatment. The behavior of outfalls discharging settled sewage is, in actual practice, much better than those discharging crude or screened sewage.

Added dilution would give lower ratios of dissolved oxygen in the sea water to the oxygen demand of the sewage. Furthermore, the degree of treatment given to the sewage is an important factor in the length of outfall sewer which will be required to keep the sewage-polluted field offshore. Further research into these phases is desirable.

Sludge deposits in the vicinity of outfalls exert high oxygen demand, and, consequently, must cripple seriously the capacity of the ocean to oxidize and dilute the sewage being discharged from the outlet. The apparently large area needed for dilution at the old Long Beach outfall could be easily explained by the high oxygen demand of the sludge, which cut down the dissolved oxygen in the sea water, limiting the organic matter that it would handle.

The authors noted a discrepancy in the actual and theoretical diameter of a rising column of sewage. It is conceivable that this may be occasioned either by the diffusion rate of salt in the sea water as it penetrates the sewage, or the swelling action may be caused by escaping dissolved gases in sewage, which at times might become as much as 20 parts per million expressed as oxygen, and might exert a material buoyant effect distorting the original line of travel. However, the ocean salt would increase dilution, and the sewage gas would shorten the time for initial dilution. There does not seem to be sufficient evidence to warrant a conclusion that the bacteria die in about 5 hours. In fact, there is evidence indicating that the time may vary from less than 1 hour to more than 5 hours. An excellent field of research would be the study of destruction of bacteria in ocean water, including such factors as temperature, oxygenation from dilution, sunlight, and time for various degrees of dilution.

The paper, as presented, has been a great help in extending knowledge on the behavior of ocean outfalls, because it places values on certain factors that heretofore were vague. For instance, in the matter of depth; even for small outfall sewers, Fig. 8 shows that 1 cu. ft. per sec. of sewage, if discharged at a depth of 30 ft., would travel horizontally, from a point above the outlet, 100 ft.; if discharged in 10 ft. of water, the distance of travel would be 2 300 ft. This certainly has a considerable bearing on locating the outfall to obtain the minimum length for the greatest depth possible.

A second point is the desirability of discharging sewage from a horizontal outlet, at a greater velocity than would ordinarily be obtained. The length of outfall can be practically cut in half by obtaining reasonable velocities of discharge. It may be that, by pumping the sewage through a 15 or 20-ft. lift and discharging it through a nozzle, the capitalized cost of pumping would be far less than the cost of an outfall twice its length.

The desirability of multiple outlets is confirmed by these experiments. In the past there has been a debate among engineers as to whether multiple outlets are actually better than single outlets. Table 4 seems to afford conclusive proof of the desirability of multiple outlets. With high-velocity discharge, they would be relatively simple to add to many existing outlets that need better dilution and more satisfactory disposal.

WARREN E. HOWLAND,* ESQ. (by letter).—The writer would like to inquire whether the experiments from which Equation (17) was derived, were conducted as illustrated by Fig. 1. That is, was the spread of the field on the surface determined "by observing the time when the edge of the field passed fixed points along the edge of the raft and along a plank cantilevered over the water at right angles to that edge"?

If t in Equation (17) refers to the time required for the colored water to reach a distance, x, from the point of discharge, it seems to the writer that the formula and the equation derived from it:

$$V = \frac{\alpha}{x^{0.50}} \qquad (37)$$

refer merely to conditions that occur when the fresh water is first discharged into the salt water rather than to conditions after the time when the flow may have become steady. Is not the condition of steady flow of greater interest than the transitory condition of initial flow? Might not the two conditions be entirely dissimilar?

The authors describe a series of observations made at Long Beach, Calif. They state that a rough check on the thickness of the field was made by "observing the time required for chips of wood to float past fixed points in one direction from the outlet". Presumably, the condition studied was more nearly that of steady flow. From these observations they conclude that:

$$V = \frac{\alpha}{x^{0.58}} \qquad (38)$$

In this case, the exponent of x is 0.58 instead of 0.50 as in Equation (37). Is this to be considered as a reasonable check, or does the difference between the two values represent an actual difference between steady flow and initial conditions, or are the experiments that lead to the two equations entirely incomparable?

The authors have not made themselves entirely clear as to the method used in computing the extent of the sewage field in the case of current flow. In the example given to illustrate Case 1, they find that in quiet water the sewage would travel 12 000 ft. before attaining a dilution of 225; but the time required to travel this distance would be 24.5 hours. In the case of current flow, the original point in the water, hypothetically, is then 44 500 ft. away from the new point where sewage is entering. Is it to be supposed that the dilutions and the relative velocities at corresponding distances from the point in the water where the sewage originally entered are the same as they would be if the sewage were still coming in at this original point?

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* Asst. Engr., State Dept. of Health, Hartford, Conn.

Apparently, the authors made this assumption when they added 12 000 and 44 500 to obtain $e = 56\,000$ ft., which is given as the total distance traveled by the sewage, presumably to attain a dilution of 225. This value of e is further corrected to 13 150 to conform to the hypothetical conditions as stated, but the alteration suggested does not seem to correct the error of the original assumption.

To state the question differently: Is not the case of sewage inflow into a moving current somewhat similar to the discharge of fresh water from a slowly moving boat? In the latter case a narrow band of brackish water is left behind and it immediately spreads sidewise, mixing with the salt water. The water at each point in the center of the spreading band is continually attaining a greater dilution by mixing as well as by actual removal of the fresh water once present. The writer believes that the contours of equal dilution, under such conditions, would be narrow ellipses or, perhaps, egg-shaped figures, but the dimensions might be difficult to compute.

The paper is of interest to sanitary engineers in Connecticut in their work of guarding health at bathing beaches.

HARRISON P. EDDY,* M. AM. SOC. C. E. (by letter).—It seems rather unfortunate that the authors chose the conditions surrounding the Deer Island Sewer Outlet in Boston Harbor to assist in corroborating their theory on predetermining the extent of a sewage field in sea water; they were evidently misinformed as to the design features of the outfall itself. The data to which they have applied their theory are in error in two respects: The depth of outlet, which they call 30 ft., and the size of outlet, which they call 18 by 30 in.

The sewage is discharged from fourteen outlet openings increasing in size toward the outer end.† The diameters of these openings, which are approximately elliptical (with the exception of the outermost, which is a 48-in. circle), vary from 25 by 44 in. to 13 by 23 in., respectively. The mean area of the fourteen openings is, therefore, 640 sq. in., instead of 540 sq. in., as assumed by the authors.

From plans of the Metropolitan Water and Sewerage Board it is found that six of the outlets are at a depth of about 50 ft. below mean low water, while the other eight openings range in depth from 30 to 46 ft., with an average depth of 38 ft. Thus, the mean depth of the fourteen outlets is 43 ft. below mean low water, instead of 30 ft.

Had the authors used the correct values for depth and area of outlet, their statements in regard to the application of the formulas to the Deer Island Outfall would probably have read as follows: The length of path should be

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† "The Metropolitan Sewerage Works," by R. W. Loud, *Journal*, Boston Soc. of Civ. Engrs., October 8, 1923, p. 325.

52 ft., and $S_0 = 42$, theoretically. For $x = 300$, $S = 262$; and for $S = 212$, $x = 195$; and the field area is 2.7 acres, or 0.025 acre per sec-ft. If the observed value ($S_0 = 21$) of the initial dilution is used, the field limit at $S = 212$ is found to be at $x = 783$ ft. and the area, 44 acres.

These results differ radically from those which were obtained by the authors and which were deemed to corroborate their theory. The initial dilution obtained by applying the formulas to the corrected data is twice that of the observed initial dilution, and the equations, therefore, are decidedly not on the safe side in this instance. However, it may well be that the authors' Equations (1) to (35) are not applicable to such a complicated outfall, in which the outlet openings are of various sizes and are located at different depths.

In order to compare the results obtained by observation with those to be expected from an application of the authors' theory, a study has been made of some tests at the Nut Island Sewer Outlet in Boston Harbor.* This outfall consists of two 60-in. pipes discharging vertically, but at the time of the tests only one outlet was in use.

In order to determine the degree of dilution that takes place and the condition of the sea water before the sewage reaches the surface, several samples of water were taken immediately over the outlet in use on October 14, 1915. Samples were collected at the same time in the line in which sewage was flowing from the outlet and at various distances therefrom, as well as samples of harbor water from points outside the path of sewage from the outlet. The tests were made when the rate of discharge was approximately 1 380 000 gal. per hour, and when the depth of water over the outlet was about 30 ft.

Observed dilutions have been derived from the free ammonia content of samples collected at various points. For example, the initial dilution (S_0) is obtained from the free ammonia content of the sewage (17.600 parts per million), the surface of the harbor above the outlet (1.753 parts per million), and the harbor water outside the path of sewage from the outlet (0.120 part per million). From these figures, $S_0 = 11$. In a similar way, the dilution near the outer limit of the sewage field, at a point 1 400 ft. from the outlet, where the free ammonia content of the water was 0.330 part per million, has been found to be 83.

The velocity of the ocean current has been estimated from the time (1.37 hours) required for floats to travel from a point 1 400 ft. from the outlet to 3 500 ft., because the limit of the sewage field, as determined by observation and analysis, seemed to be approximately 1 400 ft. from the outlet. Within the sewage field, floats of course would be influenced by the dispersion of the sewage over the harbor surface.

Table 9 gives a comparison of the results obtained by computation, using Equations (1) to (35), with those obtained by chemical analysis and observation at the Nut Island Outfall.

In the case of the Nut Island Outfall the authors' formulas yield results well on the side of safety, except for t. In the latter instance the computed value checks fairly closely with the observed value.

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* Rept. of the Mass. Dept. of Health, 1915, p. 347.

TABLE 9.—COMPARISON OF COMPUTED AND OBSERVED VALUES.

Factors.	Computed value.	Observed value.
Initial dilution, S_0	5	11
Time of travel, t, in hours, from head of rising column to edge of field	2.0	1.5
Distance, e, that sewage will travel with current, in feet	4 810	1 400

Another comparison between results obtained by observation and by the use of the authors' formulas is afforded by tests made by the State Department of Health at the sewer outlet of New Bedford, Mass., on August 9 and September 24, 1915.* The New Bedford Outfall,† which has a diameter of 60 in., discharges vertically through a single outlet at a depth of 30 ft. below low-water mark. The quantity of sewage flowing through the outlet at the time of the tests is not definitely known, but a flow of 100 gal. per capita per day has been assumed for the total population of the city in 1915 (109 568). The observed initial dilution has again been derived from free ammonia content, the determinations used, being as follows:

	Free ammonia, in parts per million.
Sewage	18.202
Sea water above outfall:	
August 9, 1915	1.276
September 24, 1915	0.207
Sea water outside area affected by sewage:	
August 9, 1915	0.110
September 24, 1915	0.044

When applied to the data given for the New Bedford Outfall, Equations (1) to (35) yield a value of $S_0 = 9$, while the values of S_0 derived from free ammonia content are 15 for August 9 and 111 for September 24. Here, again, the result obtained by the formulas is well on the side of safety. In fact, if the conditions found at the New Bedford Outlet on September 24 are at all usual, the authors' formulas would have been practically useless. On the other hand, it will be observed from the two tests cited that there is a wide variation in the value of the initial dilution above the New Bedford Outlet at different times, and many similar tests would have to be made before definite conclusions could be reached as to a fair average value of the observed initial dilution.

From the foregoing it is evident that caution must be exercised in using formulas for predetermining the extent of a sewage field in sea water, although they may well be used as guides to judgment.

A. M. RAWN‡ and H. K. PALMER,§ MEMBERS, AM. SOC. C. E. (by letter).—The writers acknowledge with thanks the discussions of their paper. Some of these discussions require no comment other than to state that they are helpful in pointing out the limitations to be set about the formulas and that they contribute additional information supplementing the original investigation or allied to it.

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* Rept., Mass. Dept. of Health, 1915.

† *Engineering News*, July 31, 1913, p. 216.

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§ Chf. Draftsman, Los Angeles County Sanitation Dists., Los Angeles, Calif.

Mr. Howland raises a question regarding the time factor used in developing the formula for spread of the sewage field. The writers state that the time used was that of the initial spread. It was found that the edge of the field advanced with the same relative velocity as that of the continuing field; that is, the velocity of flow past a given point on the raft or plank was reasonably constant for constant conditions. So long as the field thickness remains constant, which observations indicate to be true, this would necessarily be so.

It is quite impossible to obtain a series of observations yielding consistent values of the exponent of x in the equation,

$$V = \frac{F}{x^n} \quad \text{.............................(39)}$$

because the forces causing the spread of the field are small and slight disturbances materially affect the velocities.

Fig. 9 shows a series of four sets of curves drawn by assuming a field to be spreading in concentric circles of equal dilution in absolutely still water. They are drawn with the same value of C for all functions. This emphasizes clearly the effect of the exponents. In actual cases these lines would not intersect at one point, but would be parallel to the lines shown. At any distance, x, from the center the quantity of diluted water passing the ring would be QS, and the velocity would be as given by Equation (19). Combining constants, this equation becomes:

$$V = C \frac{S}{x} \quad \text{.............................(40)}$$

which involves three variables and is indeterminate unless assumptions are made.

Kinetic energy is proportional to the quantity and the square of the velocity and by properly combining all constants an equation may be written:

$$K\,E = C_1\,S\,V^2 \quad \text{.............................(41)}$$

A new variable is introduced in Equation (41) and the equation remains indeterminate. By making further assumptions, certain permissible values may be shown.

Case 1.—Assume that V is constant (see Fig. 9 (a)); then:

$$K\,E = C_2\,x \quad \text{.............................(42)}$$

and,

$$S = C_3\,x \quad \text{.............................(43)}$$

in which, x equals the horizontal distance from the center of the rising column at the surface to any point in the field. According to Equation (42), kinetic energy increases with distance, which is an impossible situation as there is no source of energy.

Case 2.—Assume that the kinetic energy is constant (see Fig. 9 (*b*)); then:

$$S = C_4\, x^{\frac{2}{3}} \quad\text{.........................}\quad (44)$$

and,

$$V = \frac{C_5}{3\sqrt{x}} \quad\text{.........................}\quad (45)$$

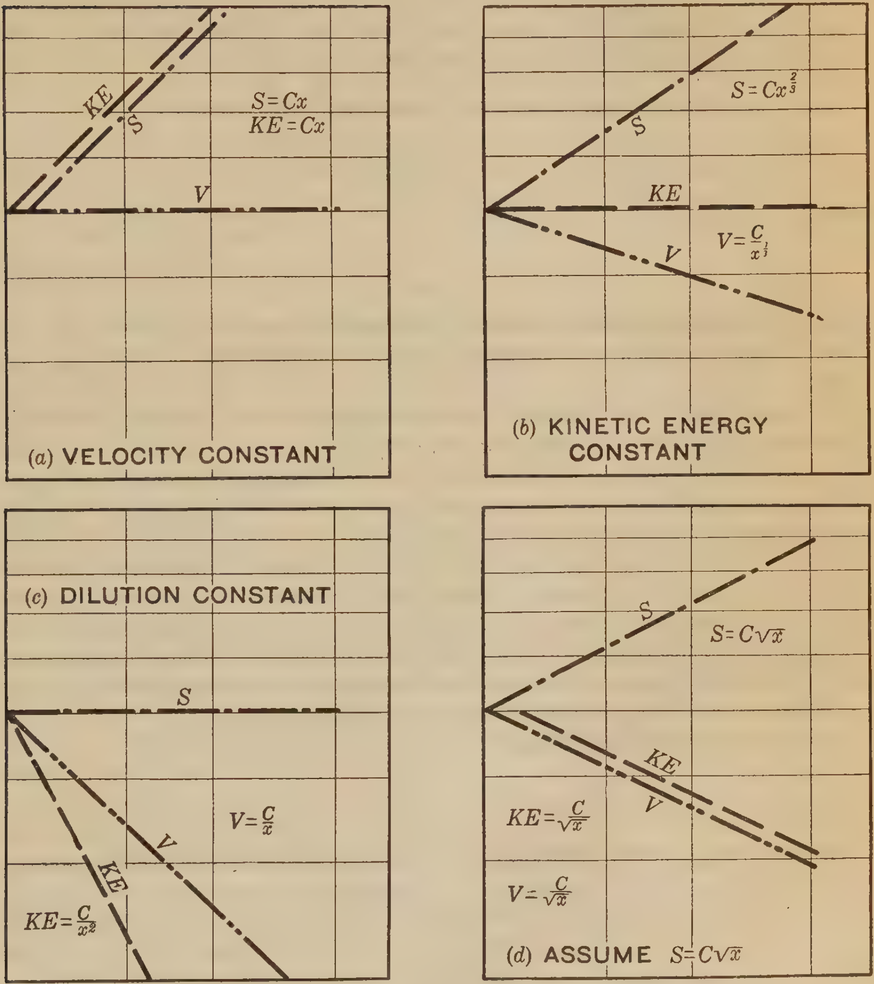

FIG. 9.—CURVES OF DILUTION, VELOCITY, AND KINETIC ENERGY.

This is an ideal condition in which there would be no loss of energy. It constitutes the upper limit for the velocity and the exponent of x lies somewhere between 0.33 for the second assumption, and 1.00 for the third assumption.

Case 3.—Assume that the dilution is constant (see Fig. 9 (*c*)); then:

$$K\,E = \frac{C_6}{x^2} \qquad (46)$$

and,

$$V = \frac{C_7}{x} \qquad (47)$$

In this instance, the kinetic energy decreases rapidly as might be expected, due to friction between the clear sea-water surface and the sewage field; but there is no dilution, which experiment disproves. It does not fit the facts and may be considered as the opposite limit to Case 2.

Case 4.—Assume that $S = C_8\sqrt{x}$ (see Fig. 9 (*d*)); then:

$$K\,E = \frac{C_9}{\sqrt{x}} \qquad (48)$$

and,

$$V = \frac{C_{10}}{\sqrt{x}} \qquad (49)$$

This is a reasonable value of velocity, and while it may not be absolutely correct, it more nearly generalizes the facts determined by the writers than any other.

In the study of the spread of actual sewage fields in sea water, the dilution was measured by the chlorine titration method and the writers feel that without extreme care and refinement, such as might be obtained in a laboratory under expert care and supervision, this method has a limiting value for determining dilutions of $S = 75$, and that the determination of values of $S > 40$, is somewhat uncertain. In Equation (22) the limiting value for $S = 212$ is obtained by substituting three known values in the equation, namely, S_0 and X_0 found from depth, quantity, and direction of discharge, and X from the bacteriological survey of the actual field at Hyperion. Thus, it is seen that $S = 212$ is not a value determined by analysis of the sewage, but that it is a computed value and is dependent upon the exponent of x.

Expressing Equation (17) as,

$$t = \frac{1}{n\,F}X^n \qquad (50)$$

and conforming to the reasoning given in the text, Equation (22) becomes,

$$S = S_0\left(\frac{x}{X_0}\right)^{2n} \qquad (51)$$

or,

$$x = X_0\left(\frac{S}{S_0}\right)^{\frac{1}{2-n}} \qquad (52)$$

Stating the value of n equal to 1.5, as in the text, this becomes,

$$x = X_0\left(\frac{S}{S_0}\right)^{2} \qquad (53)$$

but if $n = 1.6$ then,

$$x = X_0 \left(\frac{S}{S_0}\right)^{2.5} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

Again referring to the computed field at Hyperion, by use of Equation (22) the field is found to have a limiting value of $S = 212$. Had the exponent, 0.417 (corresponding to the value of $n = 1.583$), been used instead of 0.50, the value of S expressed by Equation (22) would be 135. The value, $S = 212$, was attained by assuming S_0 equals 14, and, as long as the value of S_0 is nearly this, it can be seen that it makes little difference whether the exponent 0.417 or 0.50 is used. For larger values of S_0 the x distance obtained from the use of the smaller exponent will be less than that if 0.50 is used. Therefore, the formulas and diagrams given in the text err on the side of safety, if at all, and the writers feel that their use is justified until extensive experiments offer the substitution of other coefficients and exponents. One of the best series of measurements, using nineteen floats, gave an average equation of $t = 0.097x^{1.53}$.

In determining the behavior of a sewage field influenced by an ocean current, the writers have assumed that dilution is due to the relative motions of that field and the underlying moving salt water; and that in the process of initial dilution or during the rise of the sewage to the sea-water surface the surrounding sea water will impart enough of its motion to the sewage so that the horizontal component of the rising cone just below the ocean surface will equal the velocity of the ocean current.

Mr. Eddy has pointed out the erroneous assumptions made as to the design of the Deer Island Outfall. The writers acknowledge the correction and since receiving Mr. Eddy's notes have procured a complete copy of the plans of this Outfall, as well as the descriptive paper by Mr. R. W. Loud. Despite the fact that the writers were materially in error regarding this design, their statements relating to the probable distance of travel of the sewage field are not greatly in error. In his analysis of the Outlet in operation, Mr. Eddy has incorrectly made assumptions as to the probable initial dilution and final distance of travel in calm water, having overlooked the effect of mutual interference of the jets.

Fig. 10 shows the probable pattern of cross-sections of the sewage jets, taken at the sea-water surface and at 15 ft. and 30 ft. below the surface. Interference between the jets begins to occur 30 ft. below the sea-water surface (see Fig. 11) and there is really more of the rising column surface area in contact with the surrounding water at the 30-ft. level than at either 15 ft. below, or as the combined rising columns reach the surface.

By the time the sewage reaches the surface it is substantially one flat jet (see Fig. 10(*a*)), and while the experiments and observations from which the formulas were derived, were not carried sufficiently far to assure an accurate determination of the value of S_0 for this particular case, there follows from the formula a relationship between the area of the rising column exposed to the surrounding sea water and the value of S_0 at the surface. Taking into account the merging of the columns approximately 30 ft. below the salt-water surface, and the decreased periphery at about this elevation (see Table 10) the writers

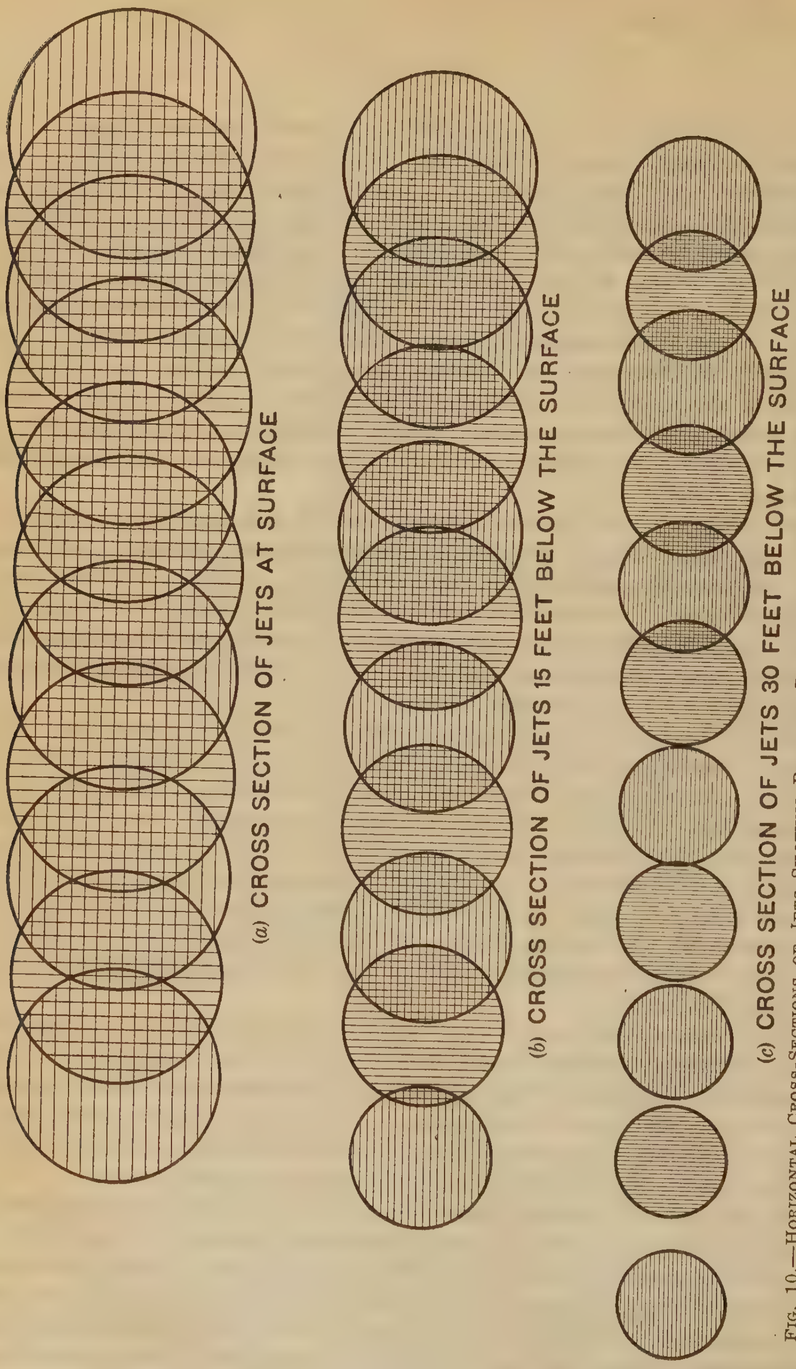
(a) CROSS SECTION OF JETS AT SURFACE
(b) CROSS SECTION OF JETS 15 FEET BELOW THE SURFACE
(c) CROSS SECTION OF JETS 30 FEET BELOW THE SURFACE

FIG. 10.—HORIZONTAL CROSS-SECTIONS OF JETS SHOWING PROBABLE BEHAVIOR OF 116 CUBIC FEET PER SECOND OF SEWAGE ISSUING FROM DEER ISLAND OUTLET DURING ITS INITIAL DILUTION.

would state that the probable value for S_0 under conditions of flow as indicated by Figs. 10 and 11 is 25 as compared to 21 used by the writers and 42 by Mr. Eddy.

TABLE 10.—COMPARISON OF ACTUAL WITH THEORETICAL PERIPHERY.

Actual periphery, in feet. (1)	Theoretical periphery, in feet. (2)	Ratio, Column (1) / Column (2). (3)	Remarks. (4)
255	684	0.37	Cross-section of jets at surface.
230	521	0.44	Cross-section of jets 15 ft. below surface.
256	354	0.72	Cross-section of jets 30 ft. below surface.

Analysis of the Deer Island Outlet indicates that 116 cu. ft. per sec. will not flow from fourteen outlets, but only the first eleven as illustrated in Fig. 11. This is also indicated in Mr. Loud's paper,* which describes an examination made in 1922, when a considerable deposit was found in the outer part of the discharge pipe, and one of the discharge openings was seen to be entirely choked with grease. An analysis of the hydraulics of the Outlet with especial reference to the value of the salt-water equivalent, will disclose why probably only eleven outlets will function under a flow of 116 cu. ft. per sec.

Table 11 is an analysis of the values of S_0 and S obtained at the Nut Island Outfall, as indicated by the ammonia and chlorine content of the samples. The table is included because of reference in Mr. Eddy's comments. It illustrates clearly the difficulties attending the study of the spread of the field, as well as of obtaining accurate samples for computation of the value of S_0. The progressive dilution in the field is erratic by either test, the chlorine content analysis seemingly giving the better results. The float velocities noted, are quite contradictory unless currents not noted by the observers were present, in that the velocity of 120 ft. per hour is obtained near the outlet where the combined velocities of sewage and bay current would be present, and 4 000 ft. per hour near the outer edge of the field where only the bay currents should be appreciable.

It is likely that, with the numerous conflicting currents in Boston Harbor, the Nut Island sewage field cannot be used to corroborate the formula for the spread of the field in calm water or where pronounced and unvarying known ocean currents are present.

The writers' experiences in obtaining samples for values of S_0 lead them to believe that the discrepancy between the observed and calculated values of this factor in Table 11, may be the result of obtaining samples at or near a ring of turbulence, surrounding the column head. This ring is indicated by a standing wave and at it the sewage undergoes a high increase in dilution; so that samples taken in or immediately outside the ring will yield erratic results.

The writers experienced great difficulty in obtaining samples directly from the head of the rising columns of sewage outlets in operation, principally because of the difficulty of rowing a boat into the column head. It is espe-

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* "The Metropolitan Sewerage Works," by R. W. Loud, *Journal*, Boston Soc. of Civ. Engrs., October 8, 1923, p. 360.

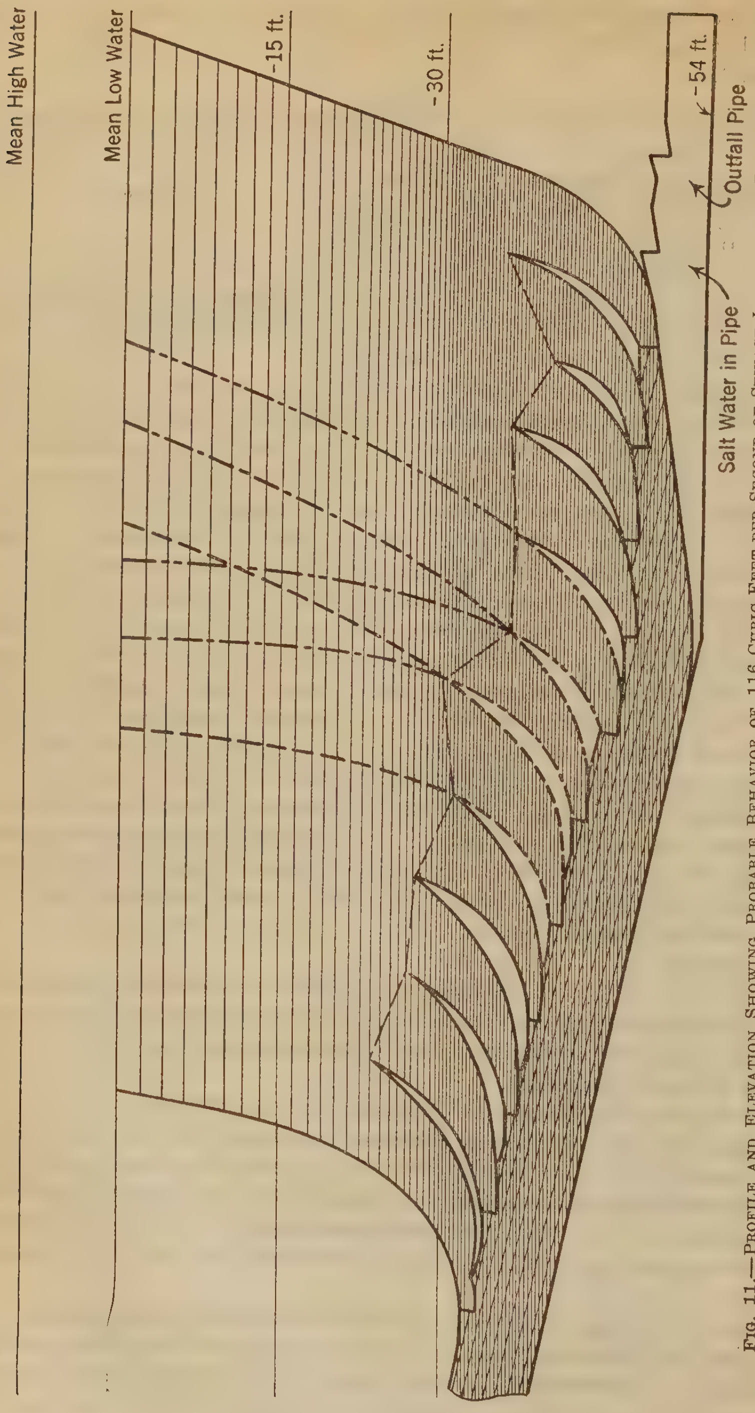
Mean High Water
Mean Low Water
-15 ft.
-30 ft.
-54 ft.
Outfall Pipe
Salt Water in Pipe

FIG. 11.—PROFILE AND ELEVATION SHOWING PROBABLE BEHAVIOR OF 116 CUBIC FEET PER SECOND OF SEWAGE ISSUING FROM DEER ISLAND OUTLET DURING ITS INITIAL DILUTION.

cially difficult to cross the ring of turbulence surrounding the column head. It is still more difficult to obtain samples from the rising column below the sea surface, and it is quite possible, even approaching a probability, to obtain specimens from either location that are extremely erratic, even if carefully taken. The rising column of sewage is not a homogeneous mass of sewage and sea water until it has moved at least a short distance from the column head. Fully 10% of all the samples taken in the experiments with small controlled outlets, was discarded by the writers as being too erratic for use.

TABLE 11.—ANALYSIS OF DILUTION AT THE NUT ISLAND OUTFALL.

Time.	Distance from outlet, in feet.	PARTS PER MILLION.		S_0.		Velocity, in feet per hour.
		Ammonia.	Chlorine.	Ammonia.	Chlorine.	
8 : 30	Sewage	17.600	535	1	1	....
9 : 13	0	1.440	15 600	13	9	....
9 : 33	0	1.950	16 200	9.5	14	....
9 : 53	0	2.400	15 400	8	8	....
10 : 13	0	1.220	15 700	16	11	....
TRAVEL OF FLOATS FROM OUTLET.		PARTS PER MILLION.		S.		Velocity, in feet per hour.
Time, in hours and minutes.	Distance, in feet.	Ammonia.	Chlorine.	Ammonia.	Chlorine.	
0	0	1.753	15 725	11	10.5	......
0 : 30	200	0.555	16 700	41	24	400
1 : 00	260	0.242	17 100	140	56	120
1 : 30	1 400	0.330	17 650	83	....	2 280
2 : 37	2 500	0.165	17 200	390	85	980
2 : 52	3 500	0.130	17 300	∞	169	4 000

Mr. Eddy refers to two tests at New Bedford, Mass., on August 9 and September 24, 1915. Depending upon the position of the tide, the theoretical value of S_0 with a flow of 17 cu. ft. per sec. would vary between 9 and 15. The test of August 9 gives a value of $S_0 = 15$ and that on September 24, $S_0 = 111$. The test of September 24 is probably the result of obtaining an extremely dilute portion of the cone which, as has been stated, is not a homogeneous mass at the time it reaches the surface. A long series of observations of this value at the New Bedford Outlet would probably disclose a great many instances departing radically from the calculated value.

Mr. Fuller's description of the operation of the diffusers on the Passaic Valley Outlet and Mr. Goudey's comments on sewage from high velocity outlets, are in line with the writers' idea of the proper design for ocean outlets. Care, however, should always be taken to prevent mutual interference of rising columns, the tendency of which must be to decrease the extremely important factor of initial dilution.

To Mr. Allen's comment upon the limitations to be set about the formula, the writers wish to add that the formulas are useless, or nearly so, unless all the factors influencing the rise and spread of the sewage field are carefully predetermined.

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Paper No. 1751

MEASURING MATERIALS FOR CONCRETE*

By William Mayo Venable,† M. Am. Soc. C. E.

Synopsis

The paper deals with those variations in workability, strength, and solidity of concrete from batch to batch, which result from unavoidable variations in the water content and gradations of the aggregates as received for use. It considers especially how the different methods of measuring the ingredients tend to increase or to reduce the difficulties of securing uniform results.

The bulking of samples of sand from twenty-two localities when measured by different methods was determined experimentally in order to ascertain the best method of measuring. Briquettes were made with all these sands, using different proportions of cement and water, in order to ascertain how the method of measurement affected the uniformity of the results. Both the strength and the solidity of these briquettes were determined.

Interesting results of these tests are here discussed, including the function and practical control of the mixing water. It is shown that the chief function of the mixing water is to make it possible to pack the other ingredients of the concrete in the least possible space; and that when the quantities of cement, sand, and stone to be used in any batch have been determined, such a quantity of water should be used as will make it possible to compact the mix into the smallest volume.

Effect of Water

Mixing of concrete has been thoroughly discussed, and the basic principles are well understood, although there remain differences of opinion as to the best procedure in applying these principles in design. However, a design depends on the properties of the materials to be used; and the greatest difficulties in the design of concrete arise from uncertainties regarding these properties.

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* Published in August, 1929, *Proceedings*.

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Concrete ready to place consists of a mechanical mixture of at least four different materials—stone or coarse aggregate, sand or fine aggregate, cement, and water. For a definite combination of perfectly dry materials and a little water only, the mixture cannot be compacted as much as when thoroughly dry. The bulk increases not because of the volume of the water, but because the water films on the grains prevent them from rolling or slipping on one another. This "bulking" at first increases with the quantity of water, until there is sufficient water to begin to lubricate the particles instead of to bind them. Thereafter an increase in water makes it possible to compact the mass more with certain mixes than can be done, even by pressure, with a perfectly dry batch. Then the voids between the grains are completely filled with water. If any more water is used, the bulk is increased accordingly.

If the quantities are measured out separately, and the fineness or the void spaces, or both, in any one of the three materials varies somewhat from batch to batch, the optimum quantity of water for maximum compactness will likewise vary. In practice it is impossible to procure stone and sand that do not vary and, consequently, there is always an element of uncertainty as to the mix. It is desirable, therefore, that the measuring methods be such as to compensate, as far as possible, for variations in the physical properties of the materials as received on the work, so that the resulting mixture has a minimum of void space to be filled by water, and has enough water—but no more than enough—to insure "workability". Such methods of measurement are available.

Most of the water used in mixing concrete is necessary only to lubricate the mass. Sometimes only 20% is required for chemical purposes, the other 80% remaining after the concrete hardens, until finally it dries out and is replaced by air. On ordinary poured concrete work, it is desirable to use only enough extra water to give maximum compaction.

Concepts of Concrete

Thus, the final mixture of stone, sand, cement, and water, may be considered in several different ways, as follows:

(*a*) A mixture of "total dry materials" with water. This does not indicate directly a desirable method of measuring each of the ingredients separately.

(*b*) A mixture of "combined aggregate" (sand and stone) with cement grout. This usually occurs first to the practical man when he attempts to interpret the water-cement ratio theory, and has not yet grasped the importance of the other materials which enter into the mix. The strength of concrete is not approximately the same as the strength of grout of the same water-cement ratio.

(*c*) A mixture of "coarse aggregate" (stone) with cement mortar. Usually, experience leads to the adoption of Method (*c*).

Bulking and Segregation

Whenever granular materials are mixed with water, two phenomena may be observed, their relative importance depending on the quantity of water used, and the conditions of placing or of compacting the mix. These are (1) bulking, and (2) segregation, due to the settling or moving of large particles faster than small ones when there is too much water.

For stone, the bulking due to dampness is negligible. For sand, it varies with the fineness and other physical properties and with the water content, sometimes amounting to 15% for relatively coarse sand, or to 50% for a very fine sand. The moisture content for maximum bulking varies between 3% and 12% of the weight of the sand. Cement behaves like very fine sand, and bulks much more than 50% with small percentages of water; but this is negligible since cement is always measured dry, usually by weighing. Sand, however, as received on the work, almost always is moist or wet, the variation seriously affecting its practical measurement.

Segregation and its effects are best shown by simple experiments. In a glass tube, say, 1½ in. in diameter and 4 ft. long, pour dry sand to a depth of about 3 in. Empty it out, fill the tube with water, and pour the sand back. The larger grains will settle faster than the smaller and the sand will reach a greater depth than in the dry.

If the same experiment is performed with cement, a similar result will be observed, except that perhaps several hours will be required for the smallest particles to settle. The cement should be mixed with some water before the final pouring, to insure that the fine and coarse grains are completely separated before settling. Allow the settled cement to harden, for a day, then remove and examine it. It will have practically no strength, the top forming merely a "mud", called "laitance", and the bottom part being soft and brittle.

Something like this occurs in mortar when an excess of water is used, but the segregation of the cement is not nearly as complete, because the distance that each particle must settle is small. A mixture of 1 volume of cement to 1.2 volumes of water makes an utterly worthless "neat" cement briquette, but a similar water-cement ratio in concrete gives considerable strength, although such a mixture is by no means recommended.

The plain fact is that to secure workability much more water is required to compact the sand and cement than to compact the cement alone. Neat cement is quite "workable" with 40% of its bulk of water, but between twice and three times this quantity is commonly required to "work" a mixture with sand and stone added.

Tamping or jarring, if it is to improve the concrete at all, does so by loosening the ingredients from positions where points rest against surfaces into positions where surfaces are in contact. It is desirable that all surplus water and air escape, leaving the grains packed together as tightly as possible, provided the water does not carry with it any cement, or cause partial segregation of the fine particles of cement from the coarse particles in the mixture. Obviously, the amount of such segregation or washing out increases with the quantity of water removed; therefore, it is desirable to use the minimum water to secure the initial condition of workability necessary for compacting the concrete. Even then the mass may not be plastic. It is only plastic when the various grains are surrounded by enough water to permit them to roll on one another without excessive surface friction.

Constant Mortar

In short, although water is necessary for workability, any excess is injurious: First, because it increases the volume; and, second, because it causes segregation of the cement. Concrete usually, although not always, is placed under conditions which render it very difficult to remove surplus water without injurious segregation. Therefore, the practical problem is how to assure the use of only the necessary quantity of water in each batch. This is really much more important than that any one material shall be kept constant in quantity, although in every respect successive batches should be as nearly alike as possible.

In making concrete, the general practice is to measure separately four materials, water, cement, sand, and stone, and to mix them. The man doing the measuring must set each measuring device to secure the correct final result. Frequently, this is difficult and it is desirable to specify the mixture in such a way that practical measurements are simple. The chief practical difficulties, with which the specifications may not have been concerned, are:

(*a*) Moisture already in the sand must either be eliminated or allowed for in measuring. This may vary excessively from batch to batch.

(*b*) The coarse aggregate may not be uniform, and, consequently, the proportion of sand to stone desired in a batch may vary. As compared with these difficulties, other problems of measurement are of minor importance.

The easiest way to overcome these difficulties practically, aside from any theoretical considerations which influence the design of the mix, is to conduct the measurement so as to produce a constant volume of uniform mortar for each batch. Then if and when necessary, the quantity of stone used therewith can be varied. Constant mortar is necessary for constant concrete, regardless of what the strength or other properties of that concrete are to be.

Sands and Their Differences

Sands commercially available for concrete work differ exceedingly in the matter of fineness and grading of particles. For the purpose of ascertaining how the physical properties affect the accuracy of various methods of measurement, the writer procured samples of commercial sands from different parts of the United States, and tested them in different ways. The sieve tests (Table 1) were conducted as follows: First, after thoroughly drying, 100 cu. cm. of each sample was measured, by pouring into a graduate, without shaking or otherwise compacting, and was weighed (Table 1, Column (3)). It was then separated by sieving, the various grades (beginning with the finest) being poured back into the graduate and the total volumes read (Columns (4) to (10)). Thus, of Sample No. 18, originally 100 cu. cm., 1.0 cu. cm. passed the finest sieve (100-mesh); 14.5 cu. cm. were retained on the next coarser sieve (48-mesh) which poured into the graduate gave a total reading of 15.5 cu. cm. The final reading was 103 cu. cm., showing that, by screening, this sand increased in volume 3 per cent.

This increase takes place with all natural sands, but differs greatly among the samples (Table 1). This is due, in large part, to differences in the shape, as well as in the size of the grains, the Pittsburgh sand with 3% increase having rounded grains, and the Boston sand with 25% increase having grains of irregular figures with sharp edges and corners.

The differences in the relative proportions of fine grains in natural sands is shown graphically for some of the samples, in Fig. 1. Although the average diameter of grains in Sample No. 6 (Table 1) is about eight times that in Sample No. 22, both are satisfactory for concrete. Evidently, the fineness sieve test alone, gives insufficient information to compare the suitability of various sands for concrete mortar.

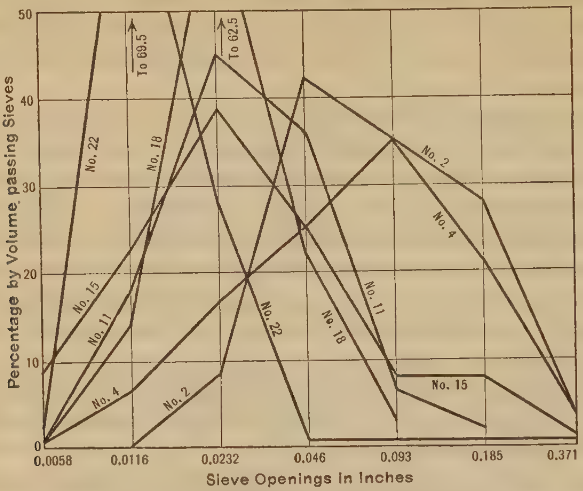

FIG. 1.—VARIATION IN SAND GRADINGS.

TABLE 1.—SIEVE TESTS OF VARIOUS SANDS.

(Total quantities, in cubic centimeters (loose), passing various openings.)

Sample No. (1)	Locality. (2)	Weight of 100 cu. cm. dry, in grammes. (3)	Sieve Openings, in Inches.						
			0.0058 (4)	0.0116 (5)	0.0232 (6)	0.046 (7)	0.093 (8)	0.185 (9)	0.371 (10)
1	St. Louis, Mo., Mississippi River	139.6	20.0	102.5	......	......	......	......	......
2	St. Louis, Mo., Mississippi River	165.3	0	trace	8.0	50.0	85.0	106.0	109.0
3	Baltimore, Md	158.3	0	2.0	16.0	50.0	80.0	102.0	......
4	Ulster County, New York (bank)	162.2	1.0	8.0	25.0	50.0	85.0	113.0	116.0
5	Baltimore, Md	166.1	0	5.0	32.0	60.0	81.0	115.0	......
6	Boston, Mass	180.5	1.0	10.0	33.0	62.0	83.0	102.0	125.0
7	Morrisville, Pa. (sand bank)	150.9	trace	5.0	43.0	82.0	97.0	109.0	110.0
8	Meramec River, Missouri	170.2	0.5	13.0	52.0	74.0	96.0	116.0	......
9	St. Louis, Mo., Mississippi River	163.9	1.5	32.0	62.0	96.0	106.0	108.0	......
10	Houston, Tex	162.9	trace	9.0	64.0	95.0	102.0	106.0	108.0
11	Lake Wales, Fla	158.6	0.2	20.0	65.0	101.0	107.0	109.0	......
12	Morrisville, Pa., Delaware River	162.0	0.5	20.0	70.0	95.0	101.0	107.0	113.0
13	Birmingham, Ala	147.7	trace	9.0	71.0	100.0	107.0	109.5	......
14	Cow Bay, N. Y	150.2	3.0	24.0	71.0	94.0	102.0	109.0	110.0
15	Boston, Mass	153.4	9.0	32.0	71.0	96.0	104.0	112.0	113.0
16	Brooklyn, N. Y. (bank)	149.2	0.5	22.0	72.0	99.0	107.0	109.0	111.0
17	Baltimore, Md	148.9	2.5	25.0	73.0	98.0	99.0	106.0	109.0
18	Pittsburgh, Pa., Allegheny River	141.6	1.0	15.5	78.0	100.0	103.0	......	......
19	Alafia River, Florida	151.2	1.0	15.0	80.0	100.0	103.0	105.0	......
20	Morgantown, W. Va	145.1	0.5	17.0	81.0	96.0	102.0	105.0	106.0
21	Baltimore, Md. (bank sand)	145.0	2.0	16.0	86.0	99.0	100.5	103.0	104.0
22	New Orleans, La., Lake Ponchartrain	146.8	2.5	72.0	100.0	100.5	100.75	102.5	103.0

The amount that sand "bulks" when moistened depends on the degree of moistness and the fineness of the sand; the finer the sand, the greater the bulking. A good idea of the extent of this bulking may be gained from Fig. 2.* The sieve analyses corresponding are given in Table 2. The Allegheny River sand, Curve *M*, is the same as Sample No. 18 in Table 1. As moisture was added the bulking effect on this Pittsburgh sand increased rapidly, reaching a maximum of 27% for 4% of water. Thus, to get the equivalent of 1 cu. ft. of dry sand requires a measuring box of 1.27 cu. ft. capacity if the sand has 4% moisture, and is measured loose. Where sand is taken from open stock piles, using volume measurement and allowing for bulking, it is generally practicable to get the specified quantity in a batch, without variation of more than 5 per cent. If, however, some of the batches are very wet or very dry, and the others moderately wet, as in a pit or bin, the variation may amount to more than 15% in the Pittsburgh District.

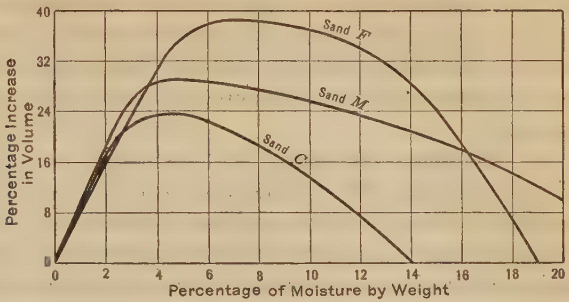

FIG. 2.—CURVES SHOWING THE BULKING OF THREE DIFFERENT SANDS DUE TO VARIOUS PERCENTAGES OF MOISTURE.

TABLE 2.—SIEVE ANALYSES OF SANDS BY WEIGHTS. PERCENTAGES RETAINED ON STANDARD SIEVES.

Sieve size.	Sand "C", Potomac River, concrete sand.	Sand "M", Allegheny River, concrete sand.	Sand "F", Potomac River, asphalt sand.
¼ in.	1	0	0
No. 10	48	13	1
No. 20	70	24	7
No. 30	79	48	19
No. 40	83	68	31
No. 50	88	89	49
No. 80	93	97	70
No. 100	95	98	82
No. 200	97	99	92
Passing No. 200..........	3	0.8	8

SATURATED SAND

However the measurement is made, great accuracy in each batch can be expected only if the moisture content is kept uniform, batch after batch.

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* "The Bulking of Moist Sands: Effect of Phenomenon on Strength and Yield of Concrete," by Arthur A. Levison, *Public Roads*, July, 1924, Vol. 5, No. 5, p. 21.

There are two attainable conditions of uniformity. One is to use perfectly dry sand; in actual work this can be attained only by artificial drying, which is expensive. The other is to have the sand saturated with water, or to measure it under water.

Even when the grading and moisture content are constant, the total quantity of sand in the container will still depend on the tamping or compacting applied to it. It may readily be made as loose as possible, by simply pouring it into the container without shaking, or as compact as possible by jarring the container.

In Table 3 will be found the results of tests of the twenty-two samples listed in Table 1, showing how the volume varies with the method of measurement when the bulking due to variable moisture is eliminated. In each case, 100 cu. cm. of perfectly dry sand was weighed (Column (2), Table 3). The sand was poured carefully into a graduate. By jarring the graduate the sand could be compacted to the volume given in Column (3), but no more. Pouring the same sand into another 100-cu. cm. graduate, half full of water, caused it to assume the volume given in Column (4), and rapping the graduate with the submerged sand caused it to compact to the volume given in Column (5). If the sand were poured suddenly into a very deep tube, filled with water, the larger grains settled first and the final volume was that given in Column (6).

TABLE 3.—VARIATION OF SAND VOLUMES AND VOIDS BY DIFFERENT MEASUREMENTS.

Sample No. (1)	Weight of 100 cu. cm. dry and loose. (2)	Volume of Sand Plus Voids, in Cubic Centimeters. Compacted by rapping. (3)	After pouring into water. (4)	Under water after rapping. (5)	Maximum segregation. (6)	Net, as determined by water displaced. (7)	Percentage of Voids. Dry. (8)	Compacted by rapping. (9)	After pouring into water. (10)	Under water after rapping. (11)	Maximum segregation. (12)	Column (10) / Column (11). (13)
1	139.6	88.0	94.0	86.0	102.0	56.0	44.0	36.4	40.4	34.9	45.1	0.864
2	165.3	98.0	108.0	98.0	116.3	70.0	30.0	28.6	35.2	28.5	39.8	0.801
3	158.3	99.0	110.0	100.0	123.0	64.0	36.0	35.3	41.8	36.0	47.5	0.861
4	162.2	99.0	114.0	102.0	123.0	66.0	34.0	33.3	42.1	35.3	46.3	0.839
5	166.1	95.0	110.0	98.0	133.7	68.0	32.0	28.4	38.2	30.6	49.3	0.801
6	180.5	97.0	112.0	100.0	124.6	70.0	80.0	27.9	37.5	30.0	43.8	0.800
7	150.9	94.0	108.0	98.0	122.8	66.0	34.0	30.0	38.8	32.6	46.2	0.842
8	170.2	96.0	107.0	94.0	120.3	66.0	34.0	31.2	38.3	30.0	45.4	0.783
9	163.9	98.0	108.0	98.0	117.6	70.0	30.0	28.6	35.2	28.6	40.8	0.802
10	162.9	94.0	110.0	98.0	120.9	68.0	32.0	27.6	38.2	27.3	45.2	0.714
11	158.6	98.0	106.0	94.0	120.5	64.0	36.0	34.7	39.6	31.3	46.7	0.790
12	162.0	96.0	108.0	98.0	119.8	66.0	34.0	31.2	38.8	32.6	44.9	0.840
13	147.7	91.0	100.0	90.0	105.8	56.0	44.0	38.0	48.8	37.7	47.1	0.732
14	150.2	91.0	110.0	98.0	127.0	62.0	38.0	31.7	43.6	36.7	51.2	0.842
15	153.4	94.0	106.0	90.0	120.5	60.0	40.0	36.1	43.4	33.3	50.2	0.767
16	149.2	94.0	104.0	94.0	114.2	60.0	40.0	36.2	42.3	36.2	47.4	0.856
17	148.9	94.0	110.0	98.0	125.0	64.0	36.0	31.9	41.8	34.7	48.8	0.830
18	141.6	90.0	100.0	90.0	107.5	58.0	42.0	35.5	42.0	35.5	46.1	0.845
19	151.2	92.0	110.0	95.0	115.8	64.0	36.0	30.4	41.8	32.6	44.7	0.780
20	145.1	95.0	102.0	92.0	117.0	60.0	40.0	36.8	41.2	34.5	48.5	0.838
21	145.0	92.0	112.0	98.0	123.9	70.0	30.0	24.0	37.5	28.5	43.4	0.733
22	146.8	95.0	108.0	90.0	112.8	58.0	42.0	38.9	46.3	35.7	48.7	0.771

Each of these five different conditions of measurement will give consistent results. That by measuring the sand loose, but completely saturated with water (Column (4)), is the simplest in practice, the measurements being reproducible on a commercial scale to within 0.5%, batch after batch, with the commercial sand used in Pittsburgh.

The net volume of sand grains only, as determined by the water displaced (Column (7)), shows that the voids vary between 30 and 44 per cent. By segregation the grains may be arranged so that more than one-half the space is void (Samples Nos. 14 and 15).

Grout to Be Added

Now suppose that just sufficient grout is to be added to fill the voids in the sand; at once the question would arise: In what way shall the voids be measured? Will the percentages in Table 3 suffice, or do these correctly represent the quantity of grout to be used? The smallest voids (when the sand is most compact) are indicated in Columns (9) and (11), and the voids of loose measurement in Columns (8) and (10). Are the voids increased beyond both these amounts when grout is added and, if so, to what extent? Without the answer to this question no determination of voids will show how much cement and water will be required to fill them properly.

Grout is not a simple liquid, but a liquid with solid grains in suspension. The amount required to "fill" the voids depends not on the voids in the sand, as measured by the water test, as much as on the volume of cement already mixed with the water to form the grout. When grout—or even dry cement—is mixed with sand, some of the particles of cement (with a diameter as large as one-eighth that of the average sand grain) prop the grains apart, thus increasing the voids to be filled, while others occupy the spaces between the grains and thus diminish the voids. Therefore, a thorough mixture of perfectly dry sand and perfectly dry cement cannot be compacted so that it will have no voids. Perhaps for any commercially available sand and cement 15% of void space is the minimum.

Moist cement and sand cannot be thoroughly compacted unless there is at least enough water to fill all the voids in the compacted mass. At least 15% of the final volume, therefore, must be water, as this is necessary to overcome surface tension among the particles and thus permit them to adjust themselves to fit snugly against one another. The quantity of water required to fill the voids completely depends not on the quantity of cement, nor on the voids in the sand measured separately by any method whatever, but on the voids in the cement and sand when mixed together in that condition of closeness permissible for "workability" for placing in the forms.

Soundness of Briquettes

To ascertain the effect of varying quantities of sand, cement, and water on the soundness and strength of mortar, a number of sample briquettes were made and tested, and the results are given in the accompanying diagrams.

As it was desired to investigate only the effect of varying the proportions, it was essential to use cement and sand of uniform quality. The cement for the briquettes came out of the same sack. All the washed Allegheny River sand was from the same stock pile, thoroughly dried and sifted. To secure uniformity of size only material passing a 0.0232-in. screen and retained on a 0.0116-in. screen was used.

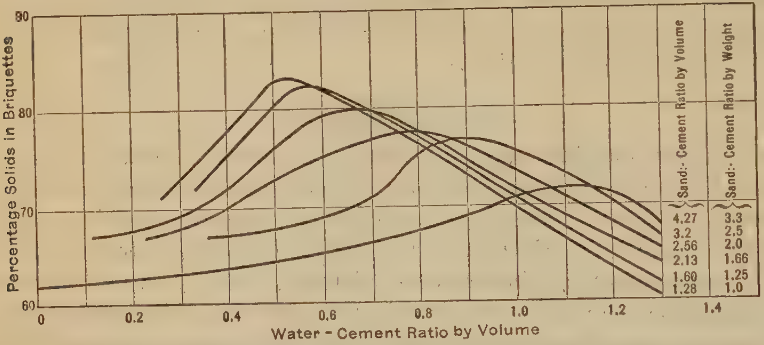

FIG. 3.—SOUNDNESS OF BRIQUETTES FOR VARIOUS RATIOS OF SAND TO CEMENT AND WATER TO CEMENT.

In turn, 100 grammes of this sand was mixed with 10 grammes, 20 grammes, etc., of cement, and with enough water to give water-cement ratios, by volume of 20%, 30%, etc. Each briquette was made in the ordinary standard mould, the materials being pressed in as tightly as possible with the fingers. It was weighed and the residual materials were weighed to check, so that the exact weights of water, cement, and sand in each briquette could be computed. It was assumed that water equal to 10% of the weight of the cement eventually combined chemically to form solid material. The percentage of solid matter to the total volume of each briquette could then be calculated.

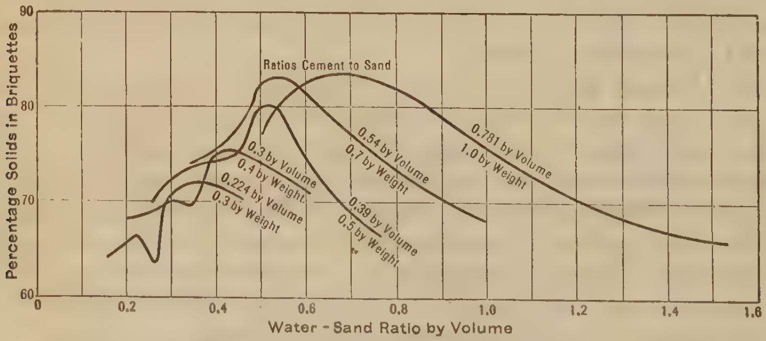

FIG. 4.—SOUNDNESS OF BRIQUETTES FOR VARIOUS RATIOS OF CEMENT TO SAND AND WATER TO SAND.

The results are given graphically in Figs. 3 and 4. With equal weights of sand and cement (Fig. 3) the soundest briquette was that containing about 52% as much cement as water, by volume. That is, it could be compacted farther in the mould than with more, or with less, water. Thus, with a sand-

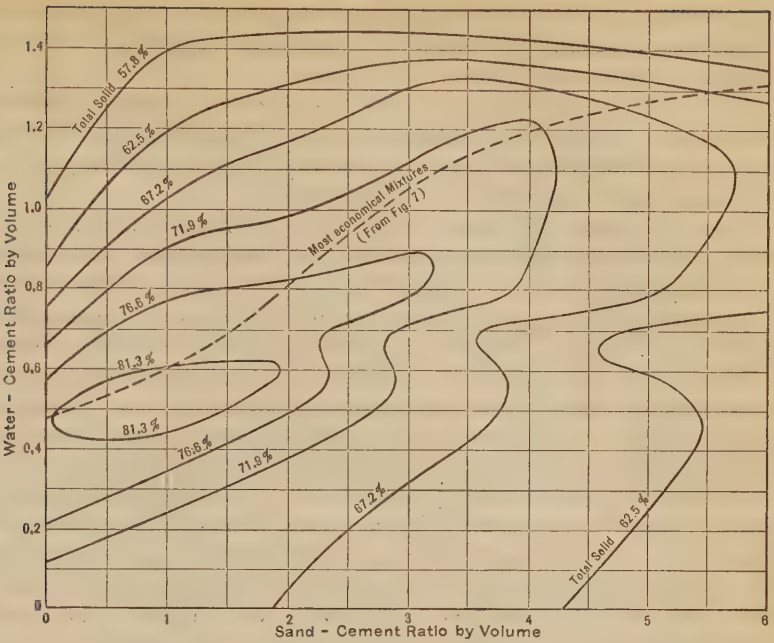

FIG. 5.—PERCENTAGE OF TOTAL SOLIDS IN BRIQUETTES OF VARIOUS MIXTURES.

cement ratio of 2 by weight, or 2.56 by volume, the water-cement ratio required to produce the densest briquette was about 0.8, while with only one-third as much cement as sand (sand-cement ratio of 4.27), it required a water-cement ratio between 1.1 and 1.2 to compact to the utmost.

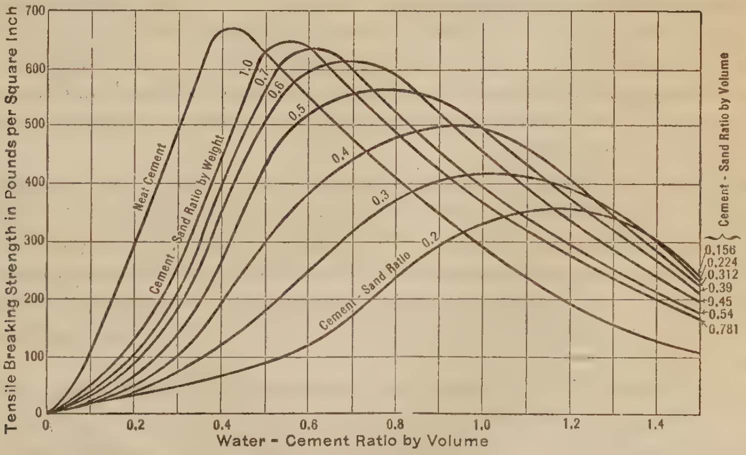

FIG. 6.—"IDEAL" CURVES SHOWING THE RELATIONSHIP BETWEEN WATER-CEMENT RATIO AND TENSILE STRENGTH FOR DIFFERENT RATIOS OF SAND TO CEMENT.

By comparison, Fig. 4 interprets the same experimental data using the water-sand ratio, instead of the water-cement ratio. Thus, a cement-sand ratio of 0.39 takes about one-half as much water as sand to make the soundest mix, but after doubling the quantity of cement, the water must be increased from 0.5 to 0.7 of the sand to get the soundest mix.

In Fig. 5, the graphs which resemble contour lines represent mixtures of equal soundness. Thus, for a mixture of 3 parts of sand to 1 part of cement, by volume, in order to make the soundest briquette, the quantity of water should be between 0.8 and 0.9 the volume of cement.

All these briquettes were tested about fifty days after moulding. Only one briquette of each mix was tested, and some of these proved to be defective. However, the results may be rationalized to allow for vagaries in the individual tests, producing the curves shown in Fig. 6. These curves compared with those in Fig. 3, indicate that there is a general parallelism between soundness and strength, as might naturally be supposed. The maximum strength, however, was obtained with a little more water than the maximum soundness, as computed, possibly because more water entered into chemical combination with cement than the 10% assumed in the calculations of soundness.

The curves in Fig. 6, as well as those in Fig. 3, are very instructive. The part of the "neat cement" curve (Fig. 6) to the right of the ordinate, 0.6, is hypothetical, as corresponding mixtures cannot be made into briquettes without the cement settling and some of the water rising to the surface. It is clear, however, that with any specified water-cement ratio there is an optimum quantity of cement and sand that will give the greatest strength.

STRENGTH TESTS

Another aspect of the same relationship is shown in Fig. 7. Discrepancies between the curves and observations are mostly for the mixes that are too wet for high-grade concrete. The strength curves in Fig. 7 should be compared with those in Fig. 5, which represent soundness. By eliminating errors by averaging tests of many briquettes, doubtless the two sets of curves would resemble one another so closely that they could be used interchangeably. In other words, maximum soundness produces maximum strength and the function of most of the mixing water is chiefly to produce workability enough to attain maximum soundness.

Both Fig. 5 and Fig. 7 show that when the sand-cement ratio is altered, the water-cement ratio should be altered also, if the sole purpose is to maintain uniform strength and soundness. For example, if a tensile strength of 460 lb. is required, and a sand-cement ratio of 2.5 and a water-cement ratio of 0.8 give too dry a mix, what should be the remedy? By increasing the water to 0.9 (Fig. 7), the mortar remains about as strong or a little stronger without adding cement.

In Fig. 7 the broken curved line represents the crest or ridge of the contours. Mixtures above this line are so wet that they may be poured about reinforcing steel with little or no tamping; those below require tamping or pressure to secure the strength recorded in the tests. This line is reproduced in Fig. 5, for comparison. Any mixture below this line will have its strength increased by adding water only; any mixture above it will increase in strength by reducing the water content, providing in each case the tamping is sufficient.

Mixtures of a specified strength will be found only within the corresponding contours. Obviously, the cheapest mix is the one requiring the least cement. Therefore, the proportions of the cheapest workable mix are found at the intersection of the broken line with the contour line representing the strength required.

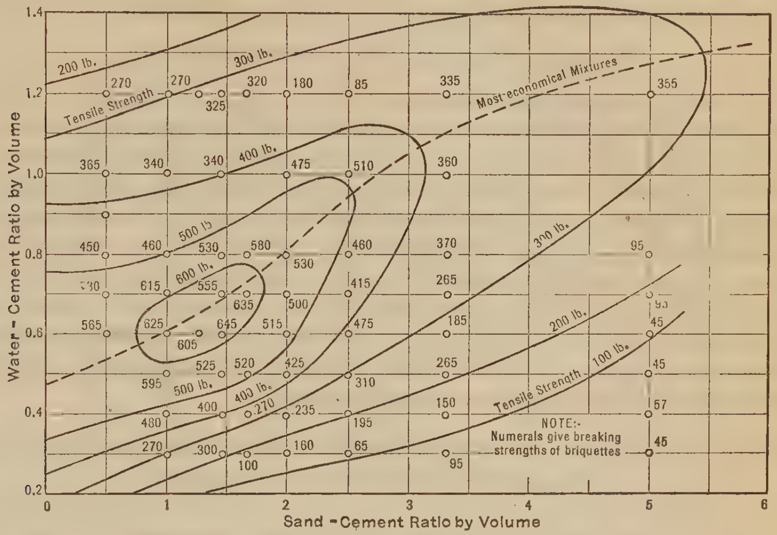

FIG. 7.—RELATIONSHIP BETWEEN SAND-CEMENT AND WATER-CEMENT RATIOS FOR BRIQUETTES OF VARIOUS TENSILE STRENGTHS.

The use of the water-cement ratio for determining the quantity of water is, of course, arbitrary. A water-sand ratio plotted from the same data (Fig. 8) is just as applicable. If the sand were supplied in perfectly dry condition, so

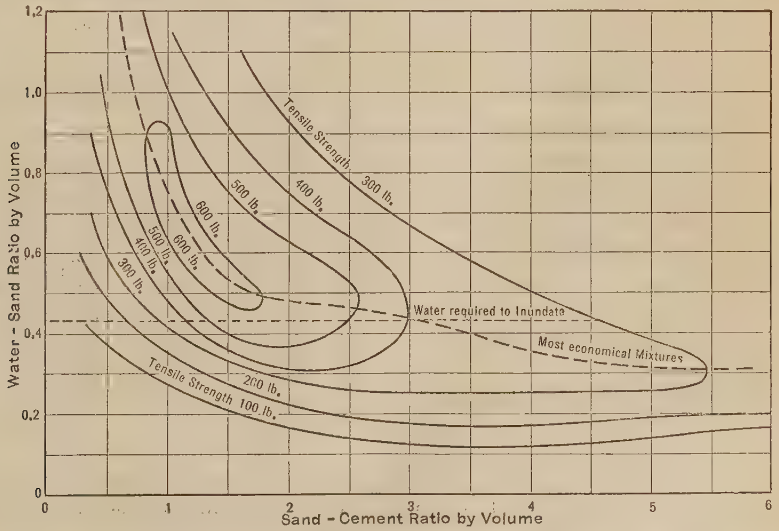

FIG. 8.—RELATIONSHIP BETWEEN SAND-CEMENT AND WATER-SAND RATIOS FOR BRIQUETTES OF VARIOUS TENSILE STRENGTHS.

that all the water may be measured separately, Figs. 7 and 8 are equally convenient. If, however, the sand is not dry, and not uniform, some advantages, as will be shown, accrue from the use of the form of Fig. 8.

All the tests on which Figs. 2 to 8 are based were made with a specially prepared sand, to secure such uniformity of materials that the effects of varying proportions could be observed, for theoretical purposes.

EFFECTS OF CONSTANT WATER-CEMENT RATIO

Obviously, the sand that, measured by bulk, requires the least cement to make a mortar of given strength and soundness is (at equal unit prices) the most economical, as cement is ordinarily the most expensive ingredient in concrete. To check the relative values of the twenty-two sand samples of Tables 1 and 3, one briquette was made with each sample. Enough water was taken to fill the voids in the sand measured inundated (Table 3, Column (10)), plus 5%, to insure perfect workability; and enough cement was used to give a water-cement ratio of 0.8 in each briquette. The sand was measured dry; the required water was calculated from the tests given in Table 3; and the volume of cement was calculated from the water, and measured by weight to avoid errors due to bulking in very small samples.

TABLE 4.—STRENGTHS OF BRIQUETTES MADE FROM VARIOUS SANDS USING A CONSTANT WATER-CEMENT RATIO OF 0.8.

Sample No. (1)	Locality. (2)	Percentage of voids, in inundated sand. (3)	Weight of 100 cu. cm. sand, in grammes. (4)	Weight of mixing water, in grammes. (5)	Weight of cement, in grammes. (6)	Percentage of bulk cement, in finished briquette. (7)	Weight of briquette, dry, in grammes. (8)	Breaking strength, in pounds per square inch. (9)	Cement-sand ratio by volume. (10)
1	St. Louis, Mo., Mississippi River	40.4	139.6	39.9	92.8	36.7	117.5	470	0.499
2	St. Louis, Mo., Mississippi River	35.2	165.3	39.9	92.8	33.3	128.5	560	0.499
3	Baltimore, Md	41.8	158.3	48.3	112.3	37.6	127.8	520	0.604
4	Ulster County, New York (bank)	42.1	162.2	50.4	117.2	37.7	128.0	655	0.630
6	Boston, Mass.	37.5	180.5	44.1	102.5	34.8	130.0	565	0.551
7	Morrisville, Pa. (sand bank)	38.8	150.9	44.1	102.5	35.7	128.3	555	0.551
8	Meramec River, Missouri	38.3	170.2	43.0	100.0	36.3	129.1	540	0.538
9	St. Louis, Mo., Mississippi River	35.2	163.9	39.9	92.8	34.8	129.1	615	0.499
10	Houston, Tex.	38.2	162.9	44.1	102.5	36.2	126.3	495	0.551
11	Lake Wales, Fla	39.6	158.6	44.1	102.5	37.2	126.3	592	0.551
12	Morrisville, Pa., Delaware River	38.8	162.0	44.1	102.5	36.7	127.1	525	0.551
13	Birmingham, Ala	43.8	147.7	46.2	107.5	40.1	125.2	545	0.578
14	Cow Bay, N. Y.	43.6	150.2	50.4	117.2	39.9	123.9	515	0.630
15	Boston, Mass.	43.4	153.4	48.3	112.3	40.0	124.6	550	0.604
16	Brooklyn, N. Y. (bank)	42.3	149.2	46.2	107.5	39.1	123.3	435	0.578
17	Baltimore, Md	41.8	148.9	48.3	112.3	38.7	124.1	585	0.604
18	Pittsburgh, Pa., Allegheny River	42.0	141.6	44.1	102.5	38.8	124.2	600	0.551
19	Alafia River, Florida	41.8	151.2	48.3	112.3	38.7	126.3	530	0.604
20	Morgantown, W. Va	41.2	145.1	44.1	102.5	38.3	123.2	545	0.551
21	Baltimore, Md. (bank sand)	37.5	145.0	44.1	102.5	35.7	124.4	590	0.551
22	New Orleans, La., Lake Ponchartrain	46.3	146.8	52.5	122.0	41.6	123.7	590	0.656

In Table 4 will be found the data for each briquette and its breaking strength after the same period of curing. As all these mortars were thoroughly "workable", they should show approximately equal strength, if maintaining a constant water-cement ratio could produce that result, regardless of the differences in the sand. From the percentages of cement to sand, by volume (Table 4, Column (10)) it will be seen that almost all these mixtures were richer than 2 of sand to 1 of cement.

TABLE 5.—STRENGTHS OF BRIQUETTES MADE FROM VARIOUS SANDS USING A CONSTANT CEMENT-SAND RATIO OF 0.4.

Sample No. (1)	Percentage of water, by volume, in sand. (2)	Water-cement ratio, by volume. (3)	Breaking strength of briquette, in pounds per square inch. (4)	Weight of dry briquette, in grammes, before breaking. (5)	Sample No. (1)	Percentage of water, by volume, in sand. (2)	Water-cement ratio, by volume. (3)	Breaking strength of briquette, in pounds per square inch. (4)	Weight of dry briquette, in grammes, before breaking. (5)
2	35.2	.88	400	129.7	12	38.8	0.98	370	124.0
4	42.1	1.05	475	126.8	13	48.8	1.22	400	118.4
6	37.5	0.94	440	125.9	14	43.6	1.09	475	124.0
7	38.8	0.96	425	122.5	15	43.4	1.08	395	121.9
8	38.3	0.94	430	127.0	16	42.3	1.06	330	119.6
9	35.2	0.88	440	127.7	17	41.8	1.05	440	122.5
10	38.2	0.95	545	124.8	20	41.2	1.03	460	118.2
11	39.6	0.99	365	122.4	21	37.5	.904	360	116.7
					22	46.3	1.16	400	116.1

CONSTANT CEMENT-SAND RATIO AND ITS RESULTS

In Table 5 are given the strengths of a series of briquettes made of a mixture of 2½ parts of submerged sand without any extra water, and 1 part of cement (both by volume), or a cement-sand ratio of 0.4; in other words, the table indicates the result of ignoring the water-cement ratio altogether, using in every case not a constant quantity of water, but the quantity necessary to saturate the sand. The water-sand ratio in Table 5, Column (2) is taken from Table 3, Column (10), and the water-cement ratio in Column (3) is computed therefrom.

In general, all these mixes are leaner in cement, than those in Table 3; they have from 80 to 60% as much cement, and less actual water, although the water-cement ratio is higher. Of course, since only one briquette was made for each test, no individual result can be considered reliable. It is remarkable, however, that the variation in breaking strengths of briquettes made by using constant proportions of inundated sand and cement, by volume, as indicated in Table 5, are so nearly alike.

NATURAL VARIATIONS IN SANDS

When sand has been measured under water, extra water may be added in any desired quantities. Table 6 gives the results of tests of briquettes all made with the same carefully screened sand, measured under water, and mixed with various quantities of cement, with extra water added in definite quantities. It shows clearly how the strength falls off usually as more water is added, when the mix is lean (Column (3)), but that when the mix is very rich (Column (6)), water must be added to make it workable enough to obtain sound briquettes.

A similar effect occurs with natural sands used in concrete, the chief difference being that there it is necessary to begin adding water with leaner mixes. These tests were made to throw light upon the effect of moderate variations in the quality and the dryness of materials in order to ascertain the best methods of measurement in the actual work—not in the laboratory only.

Now, if the sand varies in physical properties and hence in total void spaces per measured batch, an element of uncertainty is introduced. The method of measurement may be such as to magnify or minimize the importance of this; and, obviously, the latter will be the best to adopt.

Using six successive batches of Pittsburgh sand, which were dumped out and weighed after measurement, the maximum variation in the weight of the batch was 0.5%; and the maximum variation in the water content was 2 per cent. The mean variation was much less, two batches (1 800 lb.) weighing within 1 lb. of each other. Thus, in the six batches, the voids varied not more than 2 per cent.

TABLE 6.—Strength, in Pounds per Square Inch, of Briquettes Made with Inundated Sand.

Percentage of excess water.	Strength, in Pounds per Square Inch, of Briquettes Made with Inundated Sand. Percentage of Cement by Bulk.					Percentage of excess water.	Strength, in Pounds per Square Inch, of Briquettes Made with Inundated Sand. Percentage of Cement by Bulk.				
	33%	40%	50%	60%	100%		33%	40%	50%	60%	100%
(1)	(2)	(3)	(4)	(5)	(6)	(1)	(2)	(3)	(4)	(5)	(6)
0	550	560	530	635	*	30	....	....	....	450	640
6	350	445	525	560	*	36	....	....	....	....	530
12	345	465	480	....	*	42	....	....	....	....	550
18	....	300	445	....	595	48	....	....	....	....	420
24	....	....	325	465	600	54	....	....	....	....	475

* Too dry to be workable.

Water Regulation

If, however, the sand came from different sources, the variation would be greater. Then, it would be best to ascertain the extra water to be added for each sand. However, for such differences as may occur from batch to batch it is important to realize that varying the water in saturated sand in proportion to the voids is beneficial. The variation in the sands in Table 5 was extreme, but the results, except for those samples which contained deleterious materials, were remarkably uniform. Measuring the sand under water tends to produce more uniform results in the mortar than weighing the batches dry and adding uniform quantities of water.

Having determined by computation and experiment the proportions of materials that produce concrete of any desired strength and consistency, and in order to use the sand available (which may vary in moisture content, fineness, and voids), for producing concrete that is uniform, batch after batch, it is essential to:

(*a*) Eliminate the effect of moisture already in the sand on the accuracy of measurement; and,
(*b*) Regulate the quantity of water used in each batch so as to maintain the workability of the mix with as little excess water as practicable.

PRACTICAL MEASUREMENT OF SAND

Sand may be measured by bulk or by weight; moist as received on the job; dry or saturated. Some years ago the usual method was by loose bulk as received; that is, moist. There was a time when ingredients for test specimens upon which specifications were based, were measured loose and moist. For some years, however, laboratory tests have been made generally with dry rodded sand, corresponding in compactness to Column (3), Table 3, and often from 15 to 30% more compact than the same sand as received on the job. Thus, the old specification 1 : 3 : 5 with loose sand would correspond to 1 : 2.5 : 5 with dry rodded sand (laboratory conditions). Much confusion resulted and not a little injustice was done contractors by adhering to the specifications for loose measurement in the field while demanding results and costs based on rodded measurement in the laboratory.

Happily, this discrepancy has been recognized and for the most part specifications are now drawn to permit measurements on the work corresponding to those used in the laboratory. If bulk measurement is permitted, allowance for bulking of sand in the measuring containers is usually permitted also. When the bulking is determined experimentally, the error introduced is rarely more than 5% in the total quantity of sand.

Such a variation in a batch is not in itself a serious matter ordinarily, if the other materials remain constant. Thus, from Fig. 7, with the water-cement ratio constant a variation of 5% in the sand-cement ratio will have scarcely any effect on the ultimate strength. Again (Fig. 8) with the water-sand ratio constant a variation of 5% in the sand-cement ratio will have small effect on the strength. There is a more serious error, however, if the water contained in the sand varies considerably. It may be 3 to 8% of the total weight, or 7 to 20% (a difference of 13%) of the total volume of the sand, while the bulking may remain almost constant. Therefore, the water-cement ratio may easily vary by 30% when the sand-cement ratio varies only 5 per cent. Thus, the variation of 30% in the quantity of water that may enter a batch is excessive; some batches will be too wet, and others too dry. The concrete cannot be handled uniformly; and, moreover, one batch may be twice as strong as another.

The facts that only in occasional batches will the wide variations occur, and that by care in handling (avoidance of pits where sand may become water-logged) a fair degree of uniformity may be attained, render it possible to use moist bulk measurement for sand with fairly good results. On the larger and better conducted jobs the water content is checked frequently, and allowance is made in measuring additional water to be used. This, however, is subject to serious error from batch to batch.

Weighing is slightly more accurate than volume measurement, with allowance for bulking. An extreme variation of from 3 to 8% in moisture content, by weight, means only 5% in the sand; and making allowance for moisture by tests from time to time reduces the inaccuracy. Weighing makes it impossible to have those extreme variations in actual quantity of sand—15 to 30%—that may occur in bulk measurement when some of the sand is moist, but some is water-soaked, or practically dry. However, it cannot compensate for using a variable quantity of water due to ordinary fluctuations in the water content of moist sand, which are always appreciable and sometimes excessive with material exposed to the weather.

Saturating the Sand

Accurate measurement requires that sand shall be either dried out, or saturated with water at the time of measurement. This is essential, not only for accurate measurement of the sand itself, but more particularly, by ascertaining how much water goes into the measured batch with the sand, for determining the proper quantity of water to be added.

Drying sand is both troublesome and expensive, and measuring it moist introduces a probable error of 15%, or more; whereas measuring it under water gives a constant volume of sand and of water per batch. Hence, this latter method is by far the best. The additional water required must be determined, and then kept constant.

In the case of the Pittsburgh sand (Table 3, Sample No. 18), the sand measured loose under water (Column (4)) has the same bulk as if measured dry. This is true for Sample No. 13 also. Other samples, however, differ in this regard, so that it becomes necessary to determine the proportions on the basis of volume in the state actually adopted in the measuring devices, and not on any other.

The percentages of voids when sand is submerged (Table 3, Column (10)), may readily be determined. However, although this percentage varies from 35.2 to 48.8, of the twenty-two samples only two have voids exceeding 43.6 per cent. Therefore, usually the quantity of water required to saturate will be less than that in Fig. 8; and ordinarily it is necessary to add water in excess of that included with the saturated sand, in order to keep the mix workable when cement and stone are introduced. The containers used in the tests in Table 3 were small; larger apparatus would give percentages of voids somewhat less.

Batches to Capacity of Mixer

Where concrete is placed in forms, the volume turned out by the mixer frequently exceeds the amount for which forms can be prepared, and the mixer need not be run to capacity, or need be operated only part of the time. For other structures, such as massive masonry construction or concrete roads, the job capacity frequently is fixed by the mixer capacity. In any event, however, it is most economical to run the mixer at its full capacity when running at all. The best procedure in measuring materials to mix full-sized batches is as follows:

Determine from the specifications, as nearly as possible, how much sand per batch may be used to work the mixer to capacity. The measuring device should then be adjusted to fit, and such variations as are necessary to get just the mix desired should be made by adjusting the stone (measured by bulk, with or without compensation for variable voids), the cement (measured by weight), and the excess water (measured by bulk). These adjustments are much easier to make than changes in the sand-measuring device (which must be water-tight) and permit the mixer to run at nearly full capacity.

If, however, sack measurement for cement is required, the sand container must be set for the quantity corresponding with the number of sacks of cement in the batch, after determining what volume of sand in the saturated condition is compatible with the proportions specified.

Equivalence of Dry and Saturated Sands

The actual procedure must depend on how the specifications are worded, or interpreted. The method will be illustrated by assuming several different kinds of specification:

(*a*) Suppose the specifications, based on the use of saturated sand, read:

> "The sand shall be measured by bulk saturated with water, and the water filling the voids shall be included in the batch. There shall be used in each batch * * * pounds of cement for each cubic foot of sand as measured. Surplus water, constant in quantity for each batch, shall be added, as may be determined by the engineer to be necessary to secure the desired workability.
>
> "To the sand, cement, and water specified in this section shall be added the coarser aggregate or aggregates (there may be several) elsewhere described."

A specification thus drawn leaves the workability entirely in the control of the engineer. It is entirely fair and can be enforced without hardship. There will be no "over-run" or "under-run" of cement unless there is dishonesty, or the coarse aggregate departs from specifications as to voids.

(*b*) If the specification is based on laboratory tests with sand measured in a "dry rodded" condition, it may read:

> "The proportions specified for sand, cement, and water entering the various grades of concrete are based upon dry rodded sand of the grade specified (which should be the same as the sand commercially available for the work). In measuring the sand, correction must be made to insure that the actual quantity, regardless of the moisture in it, shall be equivalent in weight to the quantity specified, were it measured in a dry rodded condition."

Then it is easy for the engineer, or the contractor, to calculate the number of cubic feet of saturated sand that are exactly equivalent to dry rodded sand. The procedure is as follows:

Pack into a water pail (of Weight *a*) as much perfectly dry sand as can possibly be gotten in, by tamping or rodding. The weight of the bucket and sand, even full, will be Weight *b*. Empty out the sand so that none will be lost, pour it back slowly after the bucket has been a little more than half filled with water, allowing the surplus water to overflow. Be careful not to shake the bucket nor to waste any sand. When the bucket is exactly level full of sand and water, the weight of the remaining sand will be c and that of the pail and saturated sand will be d. Weigh the bucket full of water only (Weight e). Then,

$$\text{Weight of bucketful of rodded sand} = b - a;$$

$$\text{Weight of sand only in bucket of sand and water} = b - a - c;$$

$$\frac{\text{Volume of saturated sand to be taken}}{\text{Volume of rodded sand specified}} = \frac{b - a}{b - a - c}$$

The weight of water included in the saturated sand is $d + c - b$, and the proportion of voids in the sand (filled with water) is $\dfrac{d + c - b}{e - a}$.

If the process of measuring saturated sand is carried out properly the ratio of water to saturated sand is practically constant, whether measured in a 4-qt. pail, or in a receptacle 1 cu. yd. in capacity.

(*c*) The specifications are based on measurements of perfectly dry sand, shoveled loosely into a measuring box.

In this case, proceed exactly as in Case (*b*), using loose sand instead of rodded sand for the first two weighings.

(*d*) If the specifications state "the sand shall be measured by bulk, in the condition received on the work, loosely shoveled into a measuring box of known cubical contents", they are inaccurate. It would pay the contractor to have sand either dried or water-soaked before delivery, if such specifications are to be enforced.

(*e*) The specifications call for the sand to be weighed with or without an allowance for moisture content. Suppose that "the mixture shall consist of sand, cement, water, and stone, the sand and cement to be mixed in the proportions of 2 to 1, by weight". In this case, if saturated measurement is permitted (as it ought to be), proceed as follows:

Take a bucket, Weight a, and a quantity of perfectly dry sand, Weight b, more than sufficient to fill it. Half fill the bucket with water, and then fill it with sand until it overflows. Weigh the dry sand left over, Weight c. Finally, empty the bucket and fill it with water, calling the weight, d. Then,

$$\frac{d - a}{62.4} = \text{volume of bucket in cubic feet}$$

$$\frac{62.4\,(b - c)}{(d - a)} = \text{dry weight of 1 cu. ft. of saturated sand}$$

Let r = the specified ratio of sand to cement, by weight (in the example stated, $r = 2$). Then, for every cubic foot of saturated sand used, take $\dfrac{62.4\,(b - c)}{(d - a)\,r}$ lb. of cement.

If cement is to be measured by the sack (of 94 lb. net), for every sack of cement take $\dfrac{r \times 94 \times (d - a)}{62.4\,(b - c)}$ cu. ft. of saturated sand.

The ratio, $\frac{62.4\,(b - c)}{(d - a)}$, or its reciprocal, can be determined once for all, and it will not vary as much as the probable fluctuation in the water content of the sand as delivered on the job. Consequently, this method is more accurate than weighing the sand as the supposed specifications require. It is, in fact, a method of weighing which avoids the use of scales by employing the principle of specific gravity and approximate constancy of voids when sand is submerged.

MEASURED BY WEIGHT

Practical constructors who have used bulk cement by weighing, report that the physical properties of brands, all passing standard specifications, cause a very considerable difference in the readiness with which various cements flow through gates when being measured. The finer ground material arches more quickly under pressure and is more dusty when the arches break. When mixed with air only, the dry bulk of a given weight varies exceedingly, not only with a different, but with the same, brand of cement.

Cement differs so much in volume, according to the degree of pressure applied and time allowed in compacting, that this type of measurement should never be used. Cement should always be measured by weight. As a sack of cement is filled at the factory by automatic weighing machinery, the contents may be taken as 94 lb., and as equal to 1 cu. ft. With the sack as a unit, specifications of 1 : 2 : 4 mean 1 sack of cement, 2 cu. ft. of sand, and 4 cu. ft. of stone, or multiples of these quantities, to each batch.

With cement shipped in bulk, it is necessary to weigh out each batch independent of the cubic foot as a unit. Weighing also enables the batch to be made of a size to work the mixer to capacity. This distinct advantage recommends the weighing of cement into batches even when it is received in sacks.

THE MEASUREMENT OF STONE

Because the sand in the mortar props the stones apart, the voids as determined by water test will always be smaller than the spaces actually to be filled with mortar to produce sound concrete. Experiment will show, however, how much the necessary increase should be. The amount of mortar required depends in part on the fineness of the sand, and in part on the fineness of the stone. In any event, one thing is certain—the greater the percentage of voids in the stone the more mortar will be required to fill them; and the most economical sample of stone or gravel will have the smallest percentage of voids, provided the smallest fragments of the stone are not too small.

The stone may be measured either by bulk or by weight, for it does not vary in volume when moist, as does sand. Stone may vary considerably, however, in the shape and size of fragments, and consequently in the voids. The percentage of voids in coarse aggregate by water test may vary from a maximum of 55 to a minimum of 35 per cent. Assume, merely for illustration of the relative merits of batching by bulk and by weight, that stone received on a given job varies from 40 to 50% in voids, by water test, and weighs in the solid, 200 lb. per cu. ft.

The proportioning must be based on filling all the voids in the coarsest stone—50% of the stone volume, plus whatever increase is due to the size of the sand grains. Table 7 shows the actual voids, when the percentage of voids varies.

TABLE 7.—Volumes of Batch, Solid Stone, and Voids, in Cubic Feet.

Batch.	Percentage of Voids in Stone.		
	50	45	40
(*a*) Batch of 2 000 Lb.:			
Actual volume of batch	20	18.8	16.6
Solid stone	10	10	10
Voids	10	8.8	6.6
(*b*) Batch of 20 Cu. Ft.:			
Actual volume	20	20	20
Solid stone	10	11	12
Voids	10	9	8

Of course, there must also be some addition for the propping action of the sand grains, to be determined by experiment, but it is evident that when bulk measurement is used, and the stone has 40% voids, the batch has 2 cu. ft. more solid stone, and still is more workable than one with 50% voids. Clearly, therefore, where fixed proportions are specified, it is better to measure the stone by bulk rather than by weight, and thus to place a premium on securing the most suitable material. To measure stone by weight when it is paid for by the ton tends to produce concrete of equal cost per yard, but not necessarily of equal quality, and certainly not at the lowest expenditure in finished product for value received.

Proportioning Mortar for Varying Stone

There are several ways of varying the quantity of stone from batch to batch so as to have the batches of stone equal not in volume nor in weight, but in voids.

One is to require the stone to be screened into several grades, to measure fixed quantities of each grade separately, and finally to mix these grades. The providing of several different grades of stone at the job, and a corresponding number of separate volumetric batches has much to recommend it. In many cases, however, it is more economical in the long run to use this aggregate as it comes from the crusher or the washing plant, and to include more mortar per batch rather than to go to the expense of special preparation of stone. When it has been stored in large stock piles, there may be considerable variation in voids owing to segregation caused by the larger fragments rolling down and collecting at the bottom.

When it is not desired to separate the stone and measure each grade separately, it is practicable to allow for variations in voids by combining bulk measurement with weight measurement. The principle is this: A certain amount of mortar is sufficient for a batch of stone having maximum void spaces possible from the source of supply. Suppose, that this maximum is 50%, and that it is permissible to use such stone in a 1:2:4 mix on a given job. However, when the stone is better graded, it would be allowable to use more of it, say 1:2:4.5, because the voids are less. The problem then is, how to distinguish between the proper mixes from batch to batch as the work is carried on.

This is readily done by using a bulk measuring batcher mounted on scales. It is set for 4 parts of stone, by volume. When filled, the stone is weighed, the scales indicating automatically how much additional stone may then be introduced, by weight. On a large job quite a saving can be effected in this way. The owner's inspection is concentrated at the batching and mixing plant, instead of at the quarries, screens, and bins. Further, it is to the contractor's interest to procure the best graded stone possible because the better it is, the larger quantity he can use, with consequent saving of the more expensive material, cement.

The Size of the Job

In considering the best method of measurement for any particular piece of work, the size of the job is important. It is not always economical to go to the expense of providing accurate measuring devices, for on a small job it may cost less to allow for variability in the materials by using more cement, and by taking greater care, especially in the quantity of water used, even with crude devices. The larger the work, however, the greater will be the saving in labor, time, and the total cost of materials secured by using the most efficient and accurate devices, while the cost of installing the necessary equipment becomes proportionately less. Every project presents its own problems, the use of equipment already on hand or available being one of the factors to be considered. It is believed, however, that the point of view here presented for the very practical problem of measuring materials for batch mixtures will be helpful even in many cases where, for good reasons, it is not deemed wise to resort to the most accurate methods.

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Paper No. 1752

SILT TRANSPORTATION BY SACRAMENTO AND COLORADO RIVERS AND BY THE IMPERIAL CANAL*

By C. E. Grunsky,† Past-President, Am. Soc. C. E.

With Discussion by Messrs. C. S. Howard, Harry F. Blaney, E. S. Lindley, Thaddeus Merriman, Ivan E. Houk, and C. E. Grunsky.

Synopsis

Data relating to the transportation of silt or fine sand and other detrital material by the Sacramento and Colorado Rivers and by the Imperial Canal, with inclusion of the results of personal observations, are presented in this paper.

Repeated surveys to determine the frequent changes in the elevation of the bed of the Imperial Canal in its upper reaches, and the removal of large quantities of silt from the head reaches, have furnished an exceptional opportunity for estimating the volume of silt transported by the canal, including its bed load. Over a period of 2½ years the available data indicate deposit followed by erosion to have averaged in excess of 500 000 cu. yd. per month, with a canal flow of about 2 000 000 acre-ft. per year.

The suspended load of silt in the Colorado River has been determined with unusual care at Yuma, Ariz., by the U. S. Bureau of Reclamation, and an analysis of the observations does much to explain the distribution of silt throughout the cross-section of the river, the varying quantities of silt at different stages of the river and at different times of the year, and the total volume transported during the period of observation, since 1909.‡

Observations on Sacramento River, California

Early Gauging.—In 1879, one of the gauging stations on the Sacramento River was located at Freeport, Calif., about 14 miles below Sacramento. For this station a program was arranged by the State Engineer, for a comprehensive observation of phenomena throughout a fairly long stretch of river, to develop a dependable rating curve and to facilitate computation of discharge for any known river stage.

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* Published in August, 1929, *Proceedings*.

† Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

‡ The writer was on the Engineering Staff of the State Engineer of California from 1878 to 1888. He was a Consulting Engineer to the Imperial Irrigation District from 1916 to 1920.

The instructions covered a careful determination of the river cross-section in order that the necessity for sounding might be avoided when gaugings were repeated. Additional cross-sections, 100 ft. above and 100 ft. below the section selected for gauging with current meters, were to aid in establishing the average river dimensions over a 200-ft. float course. The gaugings were made at a fairly high stage (15 to 20 ft. above low water) and velocities were measured both with current meters and with floats.

The equipment included a catamaran, constructed of two rowboats about 2 ft. apart, united by a light platform. On this deck there was a simple structure that supported a drum of small diameter which was turned with a hand-crank and was used for lowering and raising a weighted frame that carried the current meter. With this outfit there was no trouble in lowering the meter to any desired depth and holding it there. A stay-rope from a boat 100 ft. up stream held the meter frame vertically beneath the catamaran. In taking soundings the depth was read accurately on one of the two light cables by which the frame was raised and lowered.*

The work brought its disappointments, however. There was no agreement between the original and later soundings. Evidently, something was happening to the bed of the river, and, therefore, a systematic study was undertaken to determine the extent of the changes. From day to day soundings were taken in longitudinal lines about 50 ft. apart, each carefully located. At the upper end of each line the control boat was secured by two anchors, and its position was definitely determined by triangulation from shore. A second boat was then dropped down stream and carefully held to line by a helmsman, in order to take soundings every 25 ft. The stretch of river thus covered extended 400 ft. up stream and 100 ft. down stream from the gauging section.

Hydraulic Gradients.—The cross-sections in Fig. 1 show that the configuration of the river bottom, which is a straight reach at this point, was uneven and constantly changing. The material in the river bed was clean sand, most of which was fine enough to pass a 40-mesh screen.†

The gauging work was intended not only to measure discharge, but to establish a dependable value of n in Kutter's formula. Consequently, a careful observation of the river slope was to be made. Gauge rods were set at the gauging station and at points about 4 000 ft. up stream and a like distance down stream and, for days, readings of the water-surface elevations at these three points were taken at 10-min. intervals.

When the observations were plotted on a time scale it was seen at once that the water surface gradient was constantly changing. There was a rather long rhythmic pulsation as if elongated waves were following each other down the river. It was found, in other words, that simultaneous observations would not correctly represent the river gradient, but that this gradient would have to be determined from a long series of observations. The cause of the pulsation was not discovered. In part at least it may have been the result of sudden changes in the river bed.

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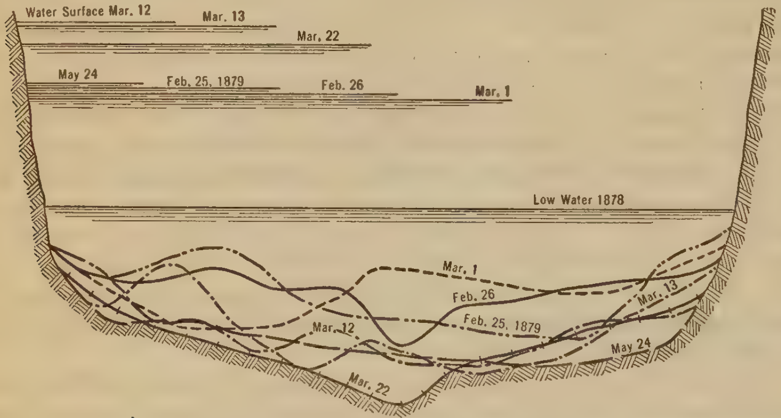
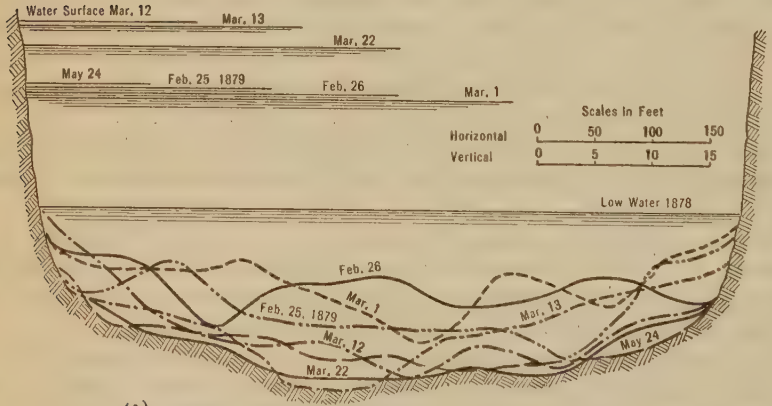
* See Report of Commr. of Public Works, California, 1895. Contribution by the writer.

† This is the writer's estimate; no actual determination was made.

River Eccentricities.—An anchored rowboat, unattended, was swamped by an accumulation of reeds and other drift on its anchor ropes. Strenuous efforts of the whole available crew, assisted by a launch, could not free it from its sand grave in the bed of the river. It was never recovered.


Water Surface Mar. 12
Mar. 13
Mar. 22
May 24
Feb. 25, 1879
Feb. 26
Mar. 1
Low Water 1878
Mar. 1
Feb. 26
Feb. 25, 1879
Mar. 13
Mar. 12
May 24
Mar. 22

(*a*) CROSS-SECTION 200 FEET ABOVE GAUGING SECTION


Water Surface Mar. 12
Mar. 13
Mar. 22
May 24
Feb. 25 1879
Feb. 26
Mar. 1
Scales in Feet
Horizontal 0 50 100 150
Vertical 0 5 10 15
Low Water 1878
Feb. 26
Mar. 1
Feb. 25, 1879
Mar. 13
Mar. 12
May 24
Mar. 22

(*b*) CROSS-SECTION 100 FEET ABOVE GAUGING SECTION

FIG. 1.—CROSS-SECTIONS IN THE SACRAMENTO RIVER AT FREEPORT, CALIF., IN 1879.

The current meters (of the Henry type), reporting every revolution by a clicking sound, indicated a decided pulsation of the current which was attributed to the effect of the whirls in the water. The whirls were quite apparent.

"Boils" were frequent at the river's highest stages. Under the influence of an upward current, the water would rise over a circular area generally 10 to 20 ft. in diameter with a maximum superelevation of about 10 in. at the center. While no cause was ascribed to these boils at the time, the writer concluded in later years that they must have been the result of a sudden yielding of a considerable area of river bottom.* This action leaves a cavity in the river bed into which the water plunges, is deflected upward, and becomes manifest at the surface as a "boil". On the Sacramento River the boils observed by the writer occurred at points where the depths were 20 to 30 ft. and at velocities in mid-stream of 5 to 7 ft. per sec.

In 1879, during a high-water stage, the piles of a dolphin above the Southern Pacific Railroad Bridge, at Sacramento, Calif., were floating away as if they had been inadequately driven. Soundings by the writer confirmed this view. They showed that in mid-stream the bed was then 25 ft. lower than it had been at a low-river stage a few months before.

In December, 1890, the Paine Crevasse occurred below Sacramento on the opposite side of the river. Soundings above this point revealed a tremendous scour due to accelerated current. More than 2 000 000 cu. yd. of sand had been scoured out of a 2-mile stretch of river within one day. In mid-stream, about opposite the down-stream edge of the crevasse, where there had been 20 ft. or more of water before the breach in the levee, the oars touched bottom in rowing; and a few days later there was again deep water at the same spot.

During the fall of 1878, in the mouth of the American River just above Sacramento, at the low stage of the river, a sand-bar was found along the north bank of the channel. During the winter freshet this sand-bar was swept out, and in the spring of 1879, during a protracted over-bank stage of the river, a new deposit was made, level from bank to bank, completely obliterating the old low-water channel. Through this new deposit the falling river cut a channel along the north bank a little later, and no trace of the low-water channel of 1878 along the south bank was left. It was estimated, at the time, that these river-bed changes represented a delivery of more than 340 000 cu. yd. of sand from the American into the Sacramento River, not including that delivered as an ordinary bed load.

Thus, under certain conditions, the configuration of fairly large sections of river bottom may be quickly changed. Every such change, moreover, evidences a rapid down-stream movement of sand in the river bed, probably by saltation of the material which, at low and ordinary water stages, travels in small sand waves.†

Mining Detritus.—The bed of the Sacramento River in its lower valley reaches has a sandy bottom. This section received large quantities of detrital matter from the hydraulic mines that were operated extensively along its tributary streams until about 1880. The accumulation below the mouth of the Feather River was large. This sand is clean and of a medium fineness. Originally, the water was fairly clear at low stages and never very heavily charged with sediment. Observations in 1879 indicated that the maximum quantity of material in suspension at mid-depth (still far in excess of that under original conditions) was about 0.1% by weight. The river, undoubtedly, carried much more material in suspension some years before, during the greatest activity of the hydraulic miners.

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* Report of Examining Commission on Rivers and Harbors for California, 1890, p. 35.

† Described by G. K. Gilbert, in *Professional Paper No. 86,* U. S. Geological Survey.

For many years the two principal tributaries of Feather River—the Yuba and Bear Rivers—were heavily charged with mining detritus. They originally entered the Great Central Valley of California in secondary valleys flanked by the higher (so-called "valley") plains. Their well-defined channels meandered through fertile bottom-lands; but the large volumes of sand dumped into the mountain reaches greatly overtaxed the transporting power of these streams. The natural channels in the secondary valleys were filled and the bottom-lands, too, were covered with a layer of clean sand. For example, in 1880 the Yuba River where it entered the main valley had obliterated its bed and had covered its bottom-lands with coarse sand 2 to 3 miles in width. It ran wild over the surface, behaving like all rivers when the supply of sand exceeds their transporting power and spreads over broad areas in shallow, unstable channels.

Why, was the first thought, had not these rivers cut deep channels and, with concentrated energy, swept the sand along instead of covering fields with it and filling houses almost to the tops of the windows? The answer is simple. The bottoms of overloaded streams are invariably flat, almost level, from bank to bank. On the other hand, streams carrying but little sand are in deep narrow channels. Whenever there is more material presented to a stream for transportation than there is capacity to transport, the stream will flatten out, increasing its width and reducing its depth. Thereby it increases the area of bottom subjected to sand traction and to scouring and correspondingly increases the volume of material pushed along as a bed load.* Many of the small streams which enter the San Joaquin Valley from the Sierra Nevada have channels of the surcharged sand type.

Progressive Bed Changes.—By the accession of sand from its principal tributaries (the Feather and the American Rivers), during the period of hydraulic mining activity, the bottom of the Sacramento River was raised progressively until about 1880. For some years thereafter the river bed near Sacramento was at a higher elevation than the water surface at low-river stages had been under natural conditions. In Fig. 2,† which shows this condition, the zero point on the gauge is 3.85 ft. above low tide in Suisun Bay, or 4.35 ft. above low tide in San Francisco Bay. From 1849 to about 1860 there was a rise and fall of the river at Sacramento due to tidal action (during the river's annual low-water period) of more than 2 ft. By 1871 the tidal range had been reduced to about 9 in., and about twenty years later had disappeared. The low-water stage at Sacramento was zero on the river gauge as late as 1856; 5.5 ft. in 1878; 7 ft. in 1880; and 8 ft. in 1889. It reached 10.5 ft. in 1890. Then bed erosion began; the material transported exceeded the supply. The low-water stage of 1894 was at 7.5 ft. on the river gauge and, by 1920, it was again at zero as in the days before hydraulic mining.

Although in the Sacramento River the low stages of recent years are due, in some measure, to the reduction of the low-water discharge by diversions for irrigation, and also to channel enlargement by dredging at down-river points for levee material, nevertheless these data indicate the passage of a great wave of débris (sand) down the river. The crest passed Sacramento about 1890 and reached the river's outfall point about 60 miles farther down stream, about 1917.

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* For discussion of detrital material delivered into the rivers of the Sierra Nevada Mountains, California, by hydraulic mines from about 1853 to 1881, see "Hydraulic Mining Débris in the Sierra Nevada," by Dr. G. K. Gilbert, *Professional Paper No. 105*, U. S. Geological Survey.

† Reproduced from Report of Examining Commission on Rivers and Harbors of California, 1890, pp. 32–33.

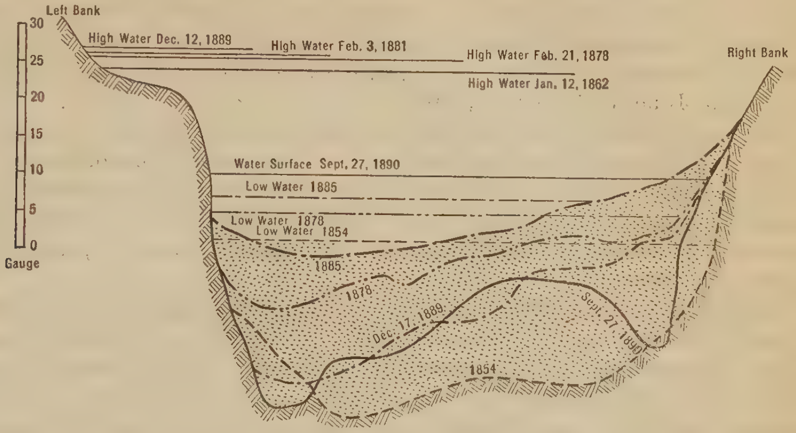

FIG. 2.—CROSS-SECTION OF THE SACRAMENTO RIVER, AT THE FOOT OF K STREET, SACRAMENTO, CALIF.

FINE SAND IN THE COLORADO

Physical Characteristics.—The rivers of the southwestern part of the United States are in a class by themselves. Of these, the Colorado is typical. At Temple Bar in 1896, about 12 miles up stream from the mouth of the Virgin River, the water looked like chocolate as it is ordinarily served. In a glass of water dipped from the surface, $\frac{1}{8}$ to $\frac{1}{4}$ in. of sediment would settle in a few minutes. A sample from the most promising mud-bar was analyzed to determine its suitability for making brick. The report showed no clay. The material was practically all silica in very fine particles.

In this part of the river there are alternate gravel-bars and deeps. In the Iceberg Canyon, a few miles farther up stream, where the bottom is a coarse gritty sand, the river is less than 250 ft. wide at its low stages and 30 to 60 ft. deep. Here and there in broad bars fine river silt is found. Some is mud and clay-like in character, shrinking and cracking as it loses moisture; some is fine sand most of which will pass through a 100-mesh sieve.

Where the river formed its great eddies the bottom was covered with this fine sediment. A loose log-boom circulated for hours in such an eddy, several hundred feet in diameter. While recapturing this boom an attempt was also made to recover a $\frac{5}{8}$-in. wire cable. Although a small scow was available and a strong pull, practically vertical, could be exerted, the cable could not be broken out of the quicksand bottom. Under water this material had a firm surface until its cohesion was broken by vibration, whereupon its behavior was momentarily that of a liquid. Over considerable areas of the bottom there were constantly occurring sudden changes of bed elevations as blocks of sand would rise and float away.

Silt Load of the Colorado River and the Imperial Canal.—The material now available for study consists of (1) determinations of the quantity of silt in suspension in Imperial Canal (which carries Colorado River water) during 1914 and 1915, by Mr. J. E. Peck; (2) determinations of bed elevation in the upper reaches of the canal, particularly from 1914 to 1920; (3) a determination of the quantity of silt in suspension in the Colorado River at Yuma, by the U. S. Reclamation Service from 1909 to date; and, finally (4), additional silt observations along the Imperial Canal and its branches, by Mr. Peck,* from June, 1917, to June, 1918.

The observations of Mr. Peck were expected to determine the amount and character of the canal's suspended load of silt. On the other hand, the soundings in the head reaches of the canal were made primarily to ascertain its constantly changing capacity to carry water. Incidentally, however, they served also in making a first estimate of the volume of material periodically deposited and eroded.

Operation of Canal.—The history of the Imperial Canal is available elsewhere.† The canal had its original head in California, at Hanlon's, about 100 yd. up stream from the International Boundary. It left the river at an oblique angle and the flow was controlled by a wooden structure. During the falling stage the head of the canal and the gate were obstructed by deposits of silt which interfered with the successful operation of the canal.

The resulting water shortage followed by failure to secure sympathetic action by Congress prompted an application to Mexico in 1904 for a permit to divert water on Mexican territory. When this privilege was granted a new connection with the river was made just below and paralleling the boundary line; but this too, was choked by silt deposits. As a further relief measure another connection was opened about 4 miles south of the International Boundary at a point where the river and canal were only about ½ mile apart. The diverted water was allowed to flow in this cut without regulating works. The result was disastrous. Erosion at an increasing rate, and, later, failure to check the increasing flow, resulted (November, 1905) in a complete deflection of the Colorado River from its original course, and the formation of the Salton Sea.‡

In the light of these facts and because silt conditions in the laterals and distributing ditches of the Imperial Canal System were giving increasing trouble, it was found desirable (1916) to provide means for silt removal from the head of the canal. Conditions both physical and political made it impracticable to construct and maintain a permanent dam in the river with a settling basin at the canal head. Therefore, suction dredges operating in the head reach of the canal were provided, one in 1918 and another in 1919. A third suction dredge was later put to work in the canal just below the Hanlon Gate. When all convenient dumping space was filled, the dredged material was returned to the river channel below the head-works. It was quickly swept away at the stream's next high stage.

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* In co-operation with C. E. Tait, U. S. Dept. of Agriculture, under the supervision of the late G. G. Anderson, M. Am. Soc. C. E., and the writer, Consulting Engineers, Imperial Irrigation District.

† *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 262; also, *loc. cit.* Vol. LIX (1907), p. 1.

‡ "Irrigation and River Control in the Colorado River Delta," by H. T. Cory, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1204.

Conditions in the Canal.—From these dredging operations in the head reach of this large canal the magnitude of the changes which must take place in the unstable bed of the Colorado River, can be inferred. Periodic surveys of the canal bed showed that at certain seasons there was a pronounced tendency to build up the canal bottom by silt deposits and at others a tendency to cut channel. During the years covered by the silt studies the flow of the canal ranged from a low-water stage in winter of somewhat more than 2 000 sec-ft. to a maximum at the height of the irrigation season—June to August—of between 5 000 and 6 000 cu. ft. per sec. The total annual diversion ranged from about 2 000 000 acre-ft. to more than 2 500 000 acre-ft. The sides and bottom of the canal to an unknown depth consist of the same fine silt that is carried as a bed and suspended load by the Colorado River.

Studies during 1917 and 1918, in co-operation with the U. S. Department of Agriculture disclosed that the material deposited in the canal and found in its bed and in the beds of its various branch canals, is fine sand. This is shown in Table 1. Attention is directed to the fact that the sampling of the material in the bed of the Main Canal at the Hanlon Gate on one occasion (July 15, 1917) showed, in relatively coarse sand, 33% caught on a No. 40 sieve.

Alignment and Structures.—At that time the Imperial Canal partook in large measure of the character of a natural stream.* The water diverted from the Colorado River was carried for 4 or 5 miles in a dredger cut and was delivered into a channel having an irregular alignment, resulting from the enlargement by erosion of an old natural overflow channel. For miles, the canal follows this "Alamo River" and has its deeps and shoals just like a natural stream. Throughout the period covered by Mr. Peck's sampling, the water entered the canal through a cut which functioned as an arm of the river. At high-water stages, generally from March to July, partition of the water from the river to the canal intake took place just as at a point of separation of a narrow from a broad delta channel. At low-water stages the flow in the river channel was checked by a temporary weir of rock or brush. Then the canal took the major portion, and occasionally all, of the river flow.

Until 1918 this water flowed in an open cut or reach about 1 700 ft. long to the Hanlon Regulating Gate, a concrete structure with eleven openings controlled by Taintor gates. In 1918, the Rockwood Head-Gate,† about 1¼ miles above the Hanlon Gate, was completed. It is a structure of concrete with seventy-five openings controlled with flash-boards. The Rockwood Gate replaces about 600 ft. of river bank, its up-stream face being set back only a few feet from the original bank alignment.

The natural turbidity of the river at the head of the Imperial Canal, it should be noted, may have been disturbed, in some measure, by the flow and sluicing control operations at the head-works of the canal of the Yuma Reclamation Project, which are at the Laguna Dam about 20 miles up stream. Before the diverted water enters the Yuma Canal, it flows through a settling

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* *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 1.

† *Loc. cit.*, Vol. 93 (1929), p. 264.

TABLE 1.—CHARACTER OF SILT DEPOSITED IN THE BEDS OF CANALS OF THE IMPERIAL CANAL SYSTEM.

FIELD DETERMINATIONS BY MR. J. E. PECK.

Date.	Number of samples averaged.	Dry weight per cubic foot, in pounds.	Specific gravity.	PERCENTAGE OF MATERIAL CAUGHT ON SIEVES.						
				No. 20.	No. 40.	No. 60.	No. 80.	No. 100.	No. 200.	Passing No. 200.
MAIN CANAL AT THE HANLON HEADING.										
March 14, 1917	3	......	2.72	0	0.05	1.22	33.07	23.54	34.38	7.17
March 28, 1917	2	......	3.64	0	0.0	0.08	5.34	17.58	59.81	17.09
May 1, 1917	6 to 12	102.	2.69	0	0.26	5.03	48.59	16.90	17.07	11.92
May 16, 1917	1	103.	......	......	......	......	......	......	......	......
July 15, 1917	5	......	2.65	12.09	20.88	33.40	20.58	1.34	4.26	1.68
CENTRAL MAIN CANAL—NEAR 10-FOOT DROP ABOUT 52 MILES FROM HEAD.										
July to December, 1917	4	98.	2.65	0	0.23	0.46	6.08	6.08	69.66	17.21
DAHLIA CANAL AT HEAD.										
July, 1917, to February, 1918	5 to 8	100.	2.68	0.02	0.22	0.44	0.76	1.70	64.47	32.15
DAHLIA NO. 12 CANAL AT HEAD.										
June, 1917, to July, 1918	5 to 8	90.	2.63	0.10	0.07	0.02	0.45	1.51	63.65	34.03
NO. 4 CANAL AT METER BRIDGE (HEADING).										
March, 1917, to February, 1918	7 to 9	94.	2.66	0.03	0.08	0.34	1.43	3.73	68.19	25.93
WEST SIDE CANAL AT BOUNDARY LINE.										
June 8, 1917	1	93.	2.67	0	0.40	2.00	10.10	17.55	47.60	21.90
NO. 8 CANAL AT METER BRIDGE NEAR HEADING.										
June, 1917, to February, 1918	3 to 7	99.	2.67	0	0	0.15	0.50	5.10	71.47	22.60
NO. 6 CANAL AT HEADING.										
July, 1917, to February, 1918	3 to 5	96.	2.63	0	4.62	3.72	6.92	1.97	50.70	31.67
NO. 5 CANAL AT HEADING.										
March, 1917, to February, 1918	4 to 7	99.	2.66	0	0.01	0.05	7.11	23.04	59.53	9.88
ROSITAS CANAL.										
June, 1917, to February, 1918	5 to 7	98.	2.65	0	0.08	0.22	14.04	5.54	61.56	14.12

basin so arranged that the deposited silt can be sluiced into the river below the dam. The sluice-gates are opened one day a week, when for the first few hours the river below is muddier than at other times. No correction due to this condition has been applied to any of the determinations of suspended silt in the canal by Mr. Peck, or in the river by the U. S. Reclamation Service.

Silt Loads.—The results of the observations on the canal are chiefly valuable in showing that the water supports and carries its suspended load throughout the entire length of the canal. Sometimes there is less, sometimes a little more, sediment in suspension at the head of the canal than at points 20 to 40 miles below; but, generally, when the water is very muddy at the head this condition persists through the entire length of the canal.

In 1914, the samples at the head of the canal were taken daily from the turbulent water just below the Hanlon Regulating Gate. The purpose was to get fair average samples from a single station in the head of the canal. The specific gravity of the silt in the canal averaged 2.64 and the voids in the material, as deposited in the bed, about 40 per cent.

A phenomenon illustrative of occasional conditions favoring the temporary conversion of the bed load of a river into a suspended load is the formation of a standing wave. This is noticed in streams of moderate depth, flowing in beds with fine sand bottom. It differs somewhat from the "boils" noted on the Sacramento River. It is a common phenomenon on the Lower Colorado. It represents a violent, sometimes fairly long-sustained attack on the river bed. "Sand is running", or "a sand wave is moving down stream", are phrases locally applied to this phenomenon. A standing wave is noted at the up-stream end of a disturbed area similar to that caused by a broad submerged snag in the river. Down stream the waves decrease in height, but extend over a gradually widening area and usually run out within 100 or 200 yd. Such a disturbance appears to the eye to be traveling slowly up stream, but is, in fact, fairly stationary. Undoubtedly, it is due to the yielding of a considerable stretch of the river bottom and the formation of a temporary depression which disturbs the steady flow of the stream. The waves gradually lose height and finally disappear when the depression is refilled to the original level by the river's bed load. The phenomenon is undoubtedly the result of the "anti-dune" movement of the stream's bed load which has been described by Dr. Gilbert.*

THE BED LOAD IN THE IMPERIAL CANAL

A Special Study.—A river's suspended load of silt can readily be determined by taking samples of the water at a sufficient number of points in any cross-section. The problem of ascertaining the bed load, however, is a difficult one to solve, and the engineer must usually be content with some random estimate.

It will not do to assume that, because somewhere else the bed load of a river was one-fifth, or some other fraction, of its suspended load, this ratio will hold in other streams. Such comparison is only permissible when all conditions are similar. There is no limit to the range of this relation. Near one extreme there are comparatively clear rivers that transport a bed load of cobbles and gravels; and at the other extreme are the streams of the Colorado type, heavily charged with fine silt flowing in beds of material differing but little from that carried in suspension. Realizing this fact, and requiring information regarding the bed load of the Imperial Canal, a special study of new and of earlier observations was made by the writer to ascertain whether changes in the elevation of the canal bottom during a number of years would "throw some light" on this subject. The stretch of canal selected extends down stream 18 000 ft. from the Hanlon Regulating Gate which, as explained, is about 1¼ miles below the Rockwood Gate in the head of the canal.

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* *Professional Paper No. 86,* U. S. Geological Survey.

For some months after the completion of the Rockwood Gate the canal water flowed from the river through this upper gate uncontrolled, but in the summer of 1918, stop-planks were put into use and thereafter the water from the river dropped over them.

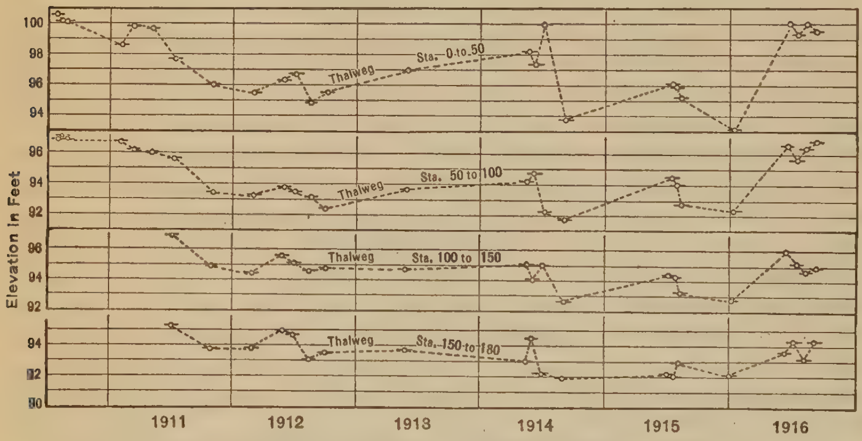

FIG. 3.—AVERAGE ELEVATION OF LINE AT GREATEST DEPTH, IMPERIAL CANAL.

Seasonal Changes.—It was found that in the spring while the river was rising from its low (winter) to a high (spring and summer) stage, more silt was deposited in the canal than its water could transport. The canal bed, at least in its upper stretches, rose to a highest position in June and was lowest in mid-winter. Considering only the selected 3.5 miles of canal, this rise, due to the deposit in the first four or five months of the year, ranged from a minimum of 1 ft. in 1912, to a maximum of 4.4 ft. in 1916. Thereupon, generally beginning in June or July, the same stretch of canal was deepened by erosion; the amount of this deepening was about 3.5 ft. in 1911, about 2.1 ft. in 1912, and about 2.7 ft. in 1915. These changes in the average elevations of the canal bed are shown diagramatically in Figs. 3 and 4. On these diagrams each point represents the average elevation of the thalweg or of three parallel lines representing the canal bottom for the particular stretch of canal as marked. These curves bring the information relating to bottom elevation forward to the time when dredging operations in the head of the canal modified the earlier conditions.

As was expected, these studies have thrown some light on the rate of the bed-load movement of the water. It appeared obvious that an increase of deposit represented accession of silt from up stream, arriving as bed load or as material dropped from suspension, and that erosion represented depletion in excess of the material arriving from up stream and temporarily coming to rest.

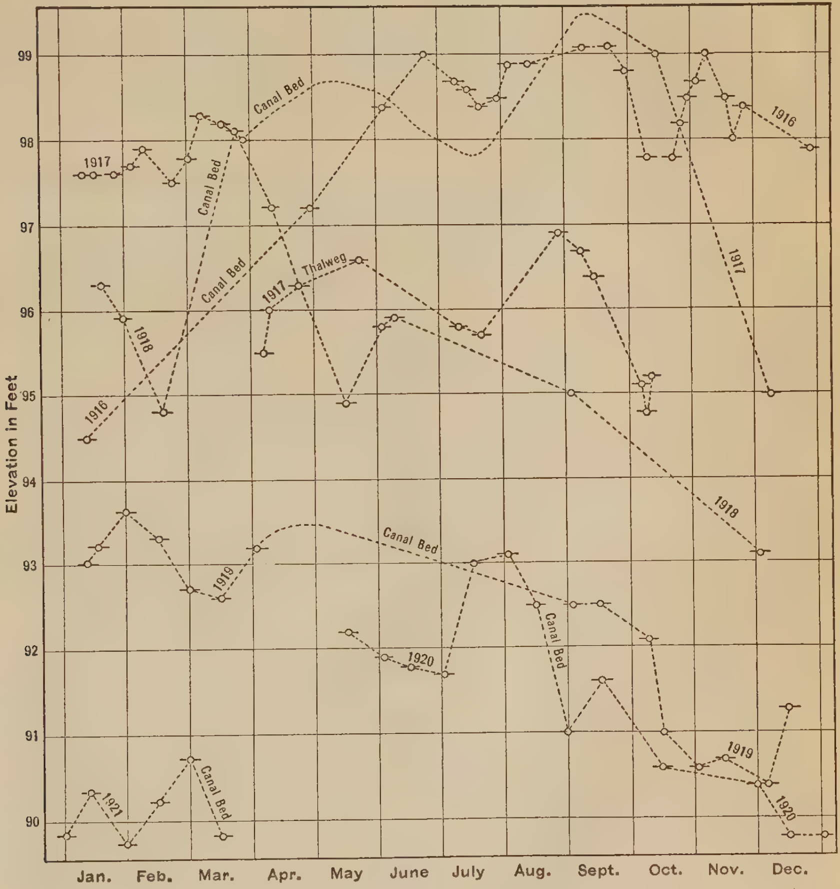

Fig. 4.—Average Elevation of Canal Bed, Imperial Canal.

These studies show that during late winter and spring at least 280 000 cu. yd. of silt per month were being deposited, temporarily, in the canal stretch under observation and that a like volume was removed later in the year. How much more silt was traveling along the bed in continuous movement in sand waves could not be ascertained, but the quantity was suspected to be large. Inspection disclosed that the canal bed, at least near shore, was covered with small sand waves about ¾ in. high and traveling slowly like sand dunes. The individual sand particles roll up and over the crests into the troughs.* Occasionally, the water would take on a disturbed appearance for a time probably denoting temporary instability of "anti-dune" sand traction.

The annual water delivery of the canal at the time of these studies was about 2 000 000 acre-ft. The water slope was about 1.5 ft. per mile, and the mean velocity about 3 to 4 ft. per sec. The silt in suspension ranged from about 0.33% to more than 1% by weight, and averaged about 0.8 per cent.

Dredging Operations.—In 1918, the canal head-works were modified, suction dredges were installed, and dredging from the canal bed was commenced. This intercepting of the silt not only improved the regimen of the canal, but also reduced the cost of canal operation by obviating the more expensive removal of the same quantity of silt from down-stream sections and from canal laterals and ditches.

TABLE 2.—REMOVAL OF SILT BY DREDGING FROM THE HEAD-WORKS SECTION OF IMPERIAL CANAL.

Period.		Cubic yards of silt excavated and removed from the canal.		Average per month, in acre-feet.
1918	July	180 000		....
	August	224 600		....
	September	248 400		....
	October	373 000		....
	November	144 700		....
	December	298 700		....
Total, six months			1 469 400	150
1919	January	286 900		....
	February	252 900		....
	March	126 200		....
	April	181 000		....
	May	353 800		....
	June	523 500		....
Total, six months			1 724 300	170
1919	July	570 100		....
	August	629 100		....
	September	594 100		....
	October	652 800		....
	November	484 000		....
	December	756 800		....
Total, six months			3 686 900	380
1920	January	798 300		....
	February	688 700		....
	March	759 900		....
	April	952 700		....
	May	886 300		....
	June	1 134 400		....
Total, six months			5 220 300	540
1920	July	1 122 800		....
	August	479 700		....
	September	342 600		....
	October	308 200		....
	November	354 300		....
	December	283 300		....
Total, six months			2 890 900	300
Grand total and average			14 991 800	310

The silt thus removed is noted in Table 2. The estimates of volume dredged were made from samples of the discharge, except during the early months of dredging when the spoil was used in filling low ground at the head of the canal to canal-bank level. On the plea of economy the amount of dredging in the head reach of the canal was reduced by the Irrigation District authorities in 1920 and was stopped with the close of that year.

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* *Professional Paper No. 86,* U. S. Geological Survey.

Despite the large aggregate volume of dredging involved (nearly 15 000 000 cu. yd. in 30 months), the effect on the regimen of the canal considered in its entirety was hardly noticeable. General conditions remained apparently unchanged; but there was some deepening of the canal bed below Hanlon's. Certainly, the dredging was beneficial and should not have stopped. It has since been resumed for a few months in the year. While it is not quite clear what effect this dredging had on the lower reaches of the canal, it is certain that if the large quantity of the removed silt had been allowed to remain in the canal it would have traveled down stream to the annoyance of the canal management and the inconvenience of the farmer who is constantly receiving excessive quantities of silt with his muddy irrigation water.

In the upper canal reaches, even before the dredging commenced, some deepening of the canal was noted. Thus, for example, the mean elevation of the canal bottom during January and February, 1918, was about 2 ft. lower than in the corresponding months of 1917. For the entire year, 1918, during the last half of which silt was removed from up-stream portions of the canal by dredging, the average lowering was also about 2 ft. In 1919, the bottom of this upper canal stretch was further deepened about 2 ft. This is to be ascribed to the depletion of the supply of silt by dredging and the continuation of bed-load movement. There was some restoration of bed elevation in 1920, the canal bottom being at about the same position in August and September as in the preceding year; and then, despite cessation of the dredging operations, the bed was lowered several feet to a position early in 1921 about 7.5 ft. lower than it had been in 1917. Some of this lowering (Figs. 3 and 4) is no doubt to be ascribed to the dredging in the head reach of the canal.

It is noteworthy that the dredging did not relieve the canal entirely of its bed load. The periodical accession and depletions of silt deposits, as evidenced by the rising and lowering of the canal bottom, continued. The dredged sumps in the head of the canal did not trap all the silt which came over the flash-boards of the Rockwood Gate. It now appears that even 500 000 cu. yd. per month (the average quantity removed for 2½ years) does not represent the entire load of fine sand carried as a bed load by the canal. How much more was being transported is not known, and no entirely satisfactory basis for estimating it has been found.

Dredging, as Table 2 shows, removed on an average about 310 acre-ft. of silt per month from the bed of the canal for 2½ years, and more than 400 acre-ft. per month for certain periods of 12 months. Had there been no removal by dredging, it is reasonable to assume that this material would have been carried down the canal mainly as a bed load.

The Bed Load of River by Comparison with That of the Canal.—It appears from the foregoing that the canal, with an annual discharge of from 2 000 000 to 2 500 000 acre-ft., transported about 4 000 acre-ft. of silt (at 100 lb. per cu. ft.) by traction, as its annual bed load. There must have been, of course, some pick-up and temporary transport in suspension. Because of this fact the estimate of canal-bed load may be accepted as a minimum.

If this determination be accepted it affords means for estimating, approximately at least, the bed load of the Colorado River itself. The traction exerted by the river on its bed may be regarded as independent of the suspended load. The sand on the bottom travels by rolling or by saltation regardless of whether the water is clear or muddy. In view of general similarity of canal and river beds, the annual bed loads of the canal and of the river may be assumed to be proportional to the annual discharge. This assumption ignores the fact that the river departs most widely from the characteristics of the canal when its discharge is greatest and the attack on its bed is most vigorous. The resulting estimate, therefore, is again more likely to be too small than too large.

On this assumption the bed load of the Colorado River, with its annual discharge of about 15 000 000 acre-ft. of water at Yuma, should be about six or seven times as great as that of the canal, or 25 000 to 30 000 acre-ft. per year. The assumption as to corrresponding fineness of material applies to the material in the beds of river and canal, as well as to that in suspension. The larger velocities in the river at its high stages, if taken into account, would probably increase somewhat its relative, and therefore, also, its total, bed load.

TABLE 3.—SILT IN SUSPENSION IN IMPERIAL CANAL, AT ITS HEAD, AND IN COLORADO RIVER, AT YUMA, ARIZONA, 1914.
(In Thousands of Tons.)

Month. (1)	IMPERIAL CANAL AT HANLON'S.				COLORADO RIVER AT YUMA.		
	Discharge, in second-feet. (2)	Weight of silt, percentage. (3)	Volume of silt at 24 hours, percentage. (4)	Silt. (5)	Discharge, in second-feet. (6)	Weight of silt, percentage. (7)	Silt. (8)
January	1 500	0.50	2.0	624	7 500	0.66	4 170
February	1 530	0.80	4.5	932	11 600	1.04	9 160
March	2 390	1.32	6.5	2 660	15 000	1.52	19 200
April	2 890	0.80	2.8	1 886	22 600	1.04	19 200
May	3 450	0.62	1.2	1 805	53 800	0.76	34 500
June	3 760	0.32	0.6	982	105 900	0.53	45 840
July	3 760	1.01	3.5	3 205	51 600	1.27	55 200
August	3 210	1.32	7.5	3 575	22 000	1.69	31 600
September	2 890	0.90	5.5	2 125	9 900	1.17	9 450
October	2 430	1.06	6.0	2 175	13 700	1.58	18 800
November	1 540	0.59	3.5	745	10 300	0.90	7 560
December	1 400	1.02	3.7	1 205	13 300	0.56	6 380
Year	2 563	0.86	....	21 927	28 100	0.93	259 860

Silt in Suspension in Imperial (Alamo) Canal.—The sampling of the water of the Imperial Canal, in 1914, was done in such a way that the results indicate the canal's full load of silt (Table 3). The samples were taken from the turbulent waters just below the gates of the Hanlon Regulator, where, by the flow under the gates, the bed load was converted temporarily into a suspended load. During that year the river carried 0.93% of its weight in suspended silt. The total silt load of the canal was 0.86 per cent. These figures indicate a smaller proportion in the canal than in the river. The difference may have been considerably more, because in addition to the 0.93% in suspension, the river must have rolled considerable material along its bed.

From July, 1918, to December, 1920, while material was being removed from the canal by dredging (Table 2), the river carried in suspension 0.87% of silt. Unfortunately, there is no corresponding determination of silt in the canal. However, that which must have entered as a suspended load, computed from the volume of water and the silt content near the surface (about 0.83%), amounted to about 1 300 acre-ft. per month at 100 lb. per cu. ft. in place. Of this, 310 acre-ft. per month were removed by dredging (Table 2). The remainder was carried down the canal in part as suspended, and in part as bed, load.

TABLE 4.—SILT IN SUSPENSION IN IMPERIAL CANAL AND IN COLORADO RIVER (1917–1918).

Month.	IMPERIAL CANAL BELOW HANLON'S.				COLORADO RIVER AT YUMA.			
	Approximate canal discharge, in second-feet.	Silt, weight, percentage.	Silt per day, in 1 000 tons.*	Silt per month, in 1 000 tons.*	Mean discharge, in second-feet.	Silt, weight, percentage.	Silt per day, in 1 000 tons.*	Silt per month, in 1 000 tons.*
1917:								
June..........	4 500	0.360	42.2	1 270	89 900	0.54	1 305	39 150
July..........	4 500	0.268	32.5	1 000	93 900	0.46	1 160	36 000
August	2 300	0.377	24.4	753	23 500	0.68	432	13 400
September	4 200	0.240	27.2	815	9 000	0.30	73	2 190
October........	3 600	0.385	37.3	1 158	7 600	0.29	59	1 840
November.....	2 300	0.113	7.1	212	7 100	0.14	27	804
December......	2 000	0.080	4.4	134	6 800	0.13	24	738
1918:								
January.......	2 200	0.088	5.2	162	6 600	0.15	27	828
February......	3 000	0.083	6.9	192	5 800	0.15	23	658
March	3 400	0.592	54.2	1 680	16 400	0.96	425	13 160
April..........	4 100	0.317	35.0	1 050	12 900	0.39	136	4 080
May............	4 400	0.265	31.4	980	29 100	0.54	423	12 100
Weighted average..........	3 470	0.280	25.8	......	23 600	0.54	344	......
Totals.........	......	......	....	9 406	......	....	....	125 868

* 2 000 lb.

The samplings of the canal water from June, 1917, to May, 1918 (Table 4) may be cited to show that part of what was suspended load in the river became bed load in the canal. These samples were taken below the zone of influence of the Taintor gates, below the Hanlon Regulator, where possibly some, if not all, separation of bed load from suspended load already had taken place. They showed 0.28% of silt in suspension, while in the same period the river water carried 0.54 per cent. Based on the mean flow of the canal (3 470 sec-ft.) and the assumption that the water which flowed unobstructed from the river into the canal was charged with the same quantity of silt as at Yuma, the total volume of silt which entered the canal was about 700 cu. yd. per month, of which 340 cu. yd. became a bed load or temporarily came to rest in the canal bed.

It should be noted, however, that at 0.54% the river water was materially short of the silt contents determined at Yuma, 0.93% for the longer period of 30 months, of which only the first 6 months are covered by simultaneous sampling.

The sampling of the disturbed or turbulent water of the canal just below the Taintor gates at Hanlon's in 1914, indicated a transport of silt by the canal water as shown in Table 3. The volume of wet silt "after 24 hours" (Column (4)), shows the variable rate at which the silt in suspension settles to the bottom of a container. The data are not intended, however, to indicate the space it will occupy when finally incorporated in the geologic structure of the river's delta region. This volume can be computed from the unit weight of the deposited material. For convenience a weight of 100 lb. per cu. ft., about the weight of the material in the beds of the main canals, has been assumed (see Table 1). As small a value as 85 lb. per cu. ft. is sometimes assumed in converting weight to volume, particularly in estimating the space that the silt which is deposited under water will occupy in a reservoir. The studies made on the Imperial Canal and its branches indicate that the larger weight is more nearly correct for this locality.

The observations in the canal, listed in Table 4, were made some distance below the Hanlon Gate and results were reported by Messrs. C. E. Tait and J. E. Peck, the sampling having been done by Mr. Peck. The observations in the Colorado River are those of the U. S. Reclamation Service. Where figures do not agree with original records it is due to the fact that it was necessary to correct averages of incomplete records by approximating missing observations. Furthermore, the values noted in this table are weighted means and not averages. The figures presented in Table 3 depart in some instances widely from those computed by the U. S. Bureau of Reclamation. This is due to the fact that the Bureau has determined monthly values as straight averages of the daily percentage determinations, while, in preparing the tables in this paper, except as otherwise noted, each daily observation was duly weighted according to the discharge to which it applies.

Variation in Silt Content.—Comparison of values in Table 3 shows that during the months of May and June, 1914, the volume of silt in suspension expressed in weight (Column (3)) was only one-half as great as the volume that settled in the bottom of a tube in 24 hours (Column (4)). In other months, February, March, September, and October, 1914, the percentage by volume exceeded that by weight sixfold. These large variations suggest lack of uniformity in the character and condition of the river's suspended load, at first attributed to the occasional large admixture of Gila River water, just above Yuma. This seemed to be borne out by the relative river stages in 1914; but, later, daily observations (June, 1917, to June, 1918, inclusive) failed to substantiate this conclusion. However, when the water was heavily

TABLE 5.—SILT IN SUSPENSION IN THE IMPERIAL (ALAMO) CANAL SYSTEM, 1917-1918.*

Miles below canal intake.	Location.	PERCENTAGE BY WEIGHT.											
		1917.							1918.				
		June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May
0.5	Main Canal, near Heading	0.36	0.27	0.58	0.24†	0.39	0.11	0.08	0.09	0.08	0.59	0.32	0.27
27.0	Main Canal, at Cudahy Check	....	....	0.68	0.41	0.41	0.13	0.15	0.13	0.14	0.66	0.41	0.41
35.0	Main Canal, at Alamo Mocho	....	....	0.62	0.28	0.38	0.15	0.12	0.11	0.12	0.59	0.38	0.37
41.0	Main Canal, at Lawrence Heading	0.40	....	....	....	0.36	0.19	0.24	0.12	0.12	0.53	0.36	0.29
48.0	East Side Canal, at Check No. 1	0.31	0.18	0.61	0.35	0.32	0.11	0.12	0.14	0.15	0.70	0.36	0.32
71.0	East Side Canal, at Myrtle Check	0.37	0.18	0.68	0.34	0.37	0.15	0.10	0.14	0.16	0.65	0.38	0.32
86.0	East Side Canal, at Junction	0.39	0.24	0.60	0.29†	0.33	0.12	0.09	0.13	....	0.55	0.23	0.34
52.0	Central Main, at 10-Ft. Drop	0.41	0.27	....	0.34	0.42	0.15	0.08	0.10	0.13	0.63	....	0.31
59.0	Dahlia Canal, at Heading	0.57	0.32	0.67	0.43	0.43	0.14	0.12	0.12	0.15	....	0.55	0.49
69.0	Dahlia Canal, at No. 12	0.48	0.28	0.62	0.35	0.40	0.11	0.10	0.09	0.08	0.47	0.34	0.27
76.0	Brawley Canal, at No. 4	0.53	0.27	0.66	....	0.28	0.14	0.07	0.10	0.11	0.62	0.43	0.34
56.0	West Side Canal, at Wisteria	....	....	0.12	0.17	0.11	0.03	0.08	0.13	0.04	0.20	0.34	0.25
65.0	West Side Canal, at Boundary	0.30	0.18	0.12	0.19	0.16	0.05	0.05	0.08	0.06	0.20	0.29	0.21
91.0	West Side Canal, at No. 8	0.38	0.23	0.18	....	0.24	0.16	0.10	0.09	0.15	0.21	0.26	0.15
104 0	Tripolium Canal, at Heading	0.31	0.28	....	....	0.23	0.10	0.07	0.08	0.12	0.23	0.30	0.28
53 0	Imp. Water Co. No. 5, at Heading	0.45	0.26	0.57	0.30	0.31	0.11	0.09	0.11	0.14	0.87	0.37	0.30
60.0	Rositas Canal, at Heading	0.53	0.30	0.52	0.34	0.57	0.23	0.22	0.19	0.19	0.68	0.41	0.48
83.0	North End Canal, at Heading	0.60	0.36	0.56	0.40	0.85	....	....	....	0.27	0.39	0.47	0.60

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* The values presented in this table are the averages of daily determinations. The field work was in charge of Mr. J. E. Peck.

† Approximations from incomplete records.

charged with silt, the settling nearly always was slower than when its load was light.

The canal does not hold quite as much silt in suspension as the river. This is ascribed to the fact that it is less turbulent than the river and, in consequence, the heavier grains of silt drop to its bed, thus becoming a part of its bed load. The difference in the suspended loads of the canal and the river was most striking in 1917 and 1918 (see Table 4), the figures being: For the canal, 0.28% and, for the river, 0.54 per cent.

Sampling of canal water was carried on systematically throughout the canal system from June, 1917, to May, 1918, the water being followed through the main canal into its many branches. The records are too voluminous to be reproduced herein in detail. The monthly summaries, however, have been assembled in Table 5.

It is to be regretted that during this period the Colorado River was never very muddy for longer than a few days. Nevertheless, Table 5 shows but slight change in the canal's suspended load of silt with distance from the intake. The suspended silt remains in suspension; the only exceptions are in the West Side laterals in which the smaller silt burden is explained by the opportunity afforded in its feeder, the Solfatara Canal, to drop a part of its load in a basin, provided for the purpose, along the northern base of Volcano Lake Levee.

Silt in Suspension in Colorado River at Yuma, Arizona

Sampling of River Water at Yuma.—Since 1909 the U. S. Reclamation Service and its successor, the U. S. Bureau of Reclamation, have made observations on the Colorado River at Yuma to determine the volume of silt transported. Twice a week samples have been taken in three verticals of a cross-section (one at mid-stream and the others at about the quarter-points). They were taken at the water surface, at mid-depth, and near the bottom of each vertical.

The results from 1909 to 1918 are presented in Table 6. The average percentage of silt carried as a suspended load during the entire ten years (0.89%) was found by comparing the total discharge with the aggregate weight of the material carried in suspension, as determined month by month. The percentages reported by the U. S. Reclamation Service for each month were accepted (for this period only) in making the calculations, although it is understood that they represent straight monthly averages instead of weighted averages. The error thus introduced for this particular period, although it is worth considering for individual months, is not material for present purposes. In Table 6, the volume of silt is computed at 100 lb. per cu. ft. The percentage of silt is as reported by the U. S. Reclamation Service. The individual determinations (8 or 9 months) appear to have been averaged—not weighted—and the monthly averages are, therefore, occasionally subject to material correction.

Distribution Throughout Year.—The large volume of silt considered in relation to discharge in January, February, and March (Table 6), is not

TABLE 6.—SILT CARRIED IN SUSPENSION BY COLORADO RIVER AT YUMA, ARIZONA, BY MONTHS, 1909–1918.

Year.	Designation.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1909	Discharge, in thousands of acre-feet	......	......	......	......	3 324	6 240	4 897	2 509	2 889	861	......	......
	Silt, percentage	......	......	......	......	0.57	0.43	0.94	0.95	1.13	0.37	......	......
	Silt, in acre-feet	......	......	......	......	11 840	16 750	28 750	14 850	20 400	1 980	......	......
1910	Discharge, in thousands of acre-feet	......	......	......	1 910	3 473	2 798	904	591	367	......	467	427
	Silt, percentage	......	......	......	0.57	0.58	0.55	0.40	0.24	0.61	......	0.36	0.28
	Silt, in acre-feet	......	......	......	6 100	12 600	9 630	2 260	652	1 390	......	1 050	745
1911	Discharge, in thousands of acre-feet	541	743	1 068	1 214	2 765	3 819	3 084	1 132	530	1 757	722	465
	Silt, percentage	1.81	1.09	1.06	0.84	0.86	0.74	1.69	1.27	0.51	2.33	0.51	0.20
	Silt, in acre-feet	2 770	5 060	7 060	350	14 870	17 600	32 250	9 000	2 680	25 600	2 300	580
1912	Discharge, in thousands of acre-feet	331	424	818	254	2 509	6 397	2 867	1 397	582	677	699	403
	Silt, percentage	0.16	0.23	0.79	.45	0.96	0.52	0.71	0.78	0.84	0.46	1.16	0.24
	Silt, in acre-feet	331	610	4 040	530	15 060	20 750	12 750	6 800	3 060	1 945	5 080	605
1913	Discharge, in thousands of acre-feet	238	337	558	1 524	2 380	2 827	1 303	580	525	635	472	393
	Silt, percentage	0.13	0.13	0.31	1.15	0.73	0.65	0.41	0.61	1.69	1.66	0.25	0.29
	Silt, in acre-feet	230	274	1 078	10 950	10 840	11 500	3 340	2 210	5 550	6 840	1 030	956
1914	Discharge, in thousands of acre-feet	462	646	923	1 364	3 308	6 575	3 168	1 350	591	840	611	818
	Silt, percentage	0.66	1.04	1.52	1.04	0.76	0.53	1.27	1.69	1.17	1.58	0.90	0.56
	Silt, in acre-feet	1 910	4 200	8 760	8 870	15 700	21 750	25 200	14 150	4 330	8 270	3 440	2 860
1915	Discharge, in thousands of acre-feet	563	1 505	951	1 789	2 941	2 892	1 897	685	270	442	356	354
	Silt, percentage	0.128	2.48	0.94	1.24	1 37	0.82	0.64	1.52	0.57	0.70	0.47	0.41
	Silt, in acre-feet	4 500	23 400	5 580	13 880	25 250	14 800	7 580	6 510	964	1 940	1 042	906
1916	Discharge, in thousands of acre-feet	2 637	1 615	2 201	2 118	3 363	3 539	2 258	1 675	736	1 636	707	454
	Silt, percentage	2.99	1.73	1.71	1.20	0.93	0.66	0.68	1.95	0.57	1.91	0.68	0.32
	Silt, in acre-feet	49 100	17 400	23 550	15 850	19 700	20 800	9 600	20 400	4 000	19 700	3 000	906
1917	Discharge, in thousands of acre-feet	562	440	603	1 565	3 030	5 350	5 775	1 442	536	465	422	420
	Silt, percentage	0.48	0.31	0.49	1.03	0.98	0.54	0.46	0.68	0.30	0.29	0.14	0.13
	Silt, in acre-feet	1 680	863	1 845	10 050	18 550	19 150	16 550	6 140	1 005	844	370	338
1918	Discharge, in thousands of acre-feet	405	323	1 008	767	1 787	3 675	2 660	710	406	474	480	451
	Silt, percentage	0.15	0.15	0.96	0.39	0.54	0.53	1.01	0.68	0.63	0.48	0.24	0.23
	Silt, in acre-feet	380	302	6 060	1 865	6 030	12 400	16 750	3 020	1 600	1 420	1 010	650
Average, 8 to 10 years	Discharge, in thousands of acre-feet	717	754	1 016	1 478	2 888	4 411	2 881	1 207	743	865	548	465
	Silt, percentage	1.70	1.39	1.14	0.93	0.83	0.59	0.86	1.11	0.97	1.41	0.59	0.33
	Silt, in acre-feet	7 600	6 520	7 240	8 600	15 000	16 350	15 520	8 340	4 490	7 590	2 035	950

fairly representative of ordinary winter conditions for the reason that in one year of the period (1916) the winter discharge of the river at Yuma was multiplied five or six times by high stages of the Gila and other lower tributaries. The admixture of these unusual freshet waters increased the average volume of silt in suspension in January, 1916, to 3% by weight, whereas ordinarily it is between 0.2% and 1.0 per cent. The discharges in February and March, to a lesser degree, were also affected. In February, 1915, the Colorado, with more than twice its ordinary discharge, had very muddy water, the silt averaging 2.48 per cent. The data for the winter months of this particular group of years, because of such extreme conditions, do not properly indicate the probable or normal relation between discharge and silt in suspension.

In the nine years, 1919 to 1927 (Table 7), all original semi-weekly observations were given weight according to river discharge in the computation of monthly mean percentages. Consequently, these are to be regarded as more nearly in accord with actual fact than those preceding 1919, particularly for the individual months.

The total quantity of silt annually transported in suspension past Yuma, estimated from these dependable figures, is shown in Table 7; also, the average volume and the probable volume of silt per calendar month for the 9-year period.

For convenience of reference the amount of silt in suspension in the Colorado River at Yuma is given in Table 8 by years. A summary of data given in Tables 6 and 7 is listed in Table 8, together with data concerning the columns of silt in suspension. The percentages from 1911 to 1918 are based on observations which were not weighted to get monthly means. The figures for these years are, therefore, subject to some correction. The percentages from 1919 to 1927 are true means and are, therefore, to be accepted as thoroughly dependable.

Comparative Amount of Silt—Surface to Bottom

Conditions Affecting Data.—The long-continued observations at Yuma are particularly valuable, not only because they furnish a dependable basis for estimating the quantity of silt in suspension, but also because they throw light on the distribution of the silt throughout a body of flowing water. The width of the Colorado River at the sampling station is about 650 ft. The verticals in which samples were taken were generally 130, 325, and 520 ft. from the left bank.

Occasionally, samples from near the bottom show excessive quantities of silt when compared with the corresponding samples at mid-depth and near the surface. These no doubt reflect a momentary condition of bed-load saltation and should be connected with the river's bed load rather than with its suspended load. In the following deductions such clearly discordant determinations of silt in suspension have been disregarded.

The Yuma Reclamation Project Canal taps the river at Laguna Dam about 12 miles up stream from Yuma. The head-works of the canal at this point include a desilting arrangement already described. When the tem-

TABLE 7.—SILT CARRIED IN SUSPENSION BY COLORADO RIVER AT YUMA, ARIZONA, BY MONTHS, 1919-1927.

(Volume of silt computed at 100 lb. per cu. ft. All results of daily observations have been weighted.)

Year.	Designation.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1919	Discharge, in thousands of acre-feet	231	398	544	1 230	2 220	2 040	1 240	658	307	325	607	947
	Silt, percentage	0.16	0.34	0.86	0.97	0.50	0.55	2.04	2.54	1.10	7.78	0,54	1.59
	Silt, in acre-feet	231	846	2 920	7 460	6 940	7 012	12 810	10 460	2 110	1 584	2 049	9 410
1920	Discharge, in thousands of acre-feet	701	2 190	1 110	1 210	2 840	7 680	2 650	1 090	502	400	619	452
	Silt, percentage	0.54	1.74	1.68	1.08	0.88	0.62	0.69	0.87	(0.76)	0.24	0.40	0.21
	Silt, in acre-feet	2 366	23 800	11 660	8 166	15 620	30 460	11 430	5 926	(2 384)	600	1 548	593
1921	Discharge, in thousands of acre-feet	427	409	824	815	2 670	6 600	2 820	2 160	1 070	552	448	633
	Silt, percentage	0.24	0.18	0.58	0.34	0.76	0.50	0.74	2.34	2.32	0.45	0.33	0.79
	Silt, in acre-feet	640	460	2 986	1 731	12 680	20 620	13 040	31 600	15 520	1 552	924	3 124
1922	Discharge, in thousands of acre-feet	799	600	996	1 140	3 440	5 810	1 940	744	524	240	326	455
	Silt, percentage	0.93	0.63	1.51	1.08	1.10	0.59	0.57	1.19	1.19	0.17	0.22	0.27
	Silt, in acre-feet	4 646	2 361	9 404	7 695	23 640	21 410	6 910	5 535	3 898	255	448	768
1923	Discharge, in thousands of acre-feet	328	321	540	1 080	2 880	5 070	2 620	1 510	1 270	736	828	666
	Silt, percentage	0.19	0.18	0.37	0.86	0.72	0.61	0.79	2.00	3 17	1.67	1.41	0.85
	Silt, in acre-feet	390	361	1 249	5 806	12 950	19 330	12 930	18 870	25 160	7 690	7 298	3 536
1924	Discharge, in thousands of acre-feet	785	481	520	1 316	2 563	3 195	1 123	288	184	244	369	355
	Silt, percentage	1.45	0.45	0.44	1.39	0.86	0.76	0.62	0.66	0.75	0.71	0.34	0.25
	Silt, in acre-feet	7 112	1 352	1 430	11 430	13 780	15 170	4 353	1 188	862	1 082	784	544
1925	Discharge, in thousands of acre-feet	212	349	531	1 090	1 780	2 498	1 740	738	1 210	1 150	677	474
	Silt, percentage	0.32	0.33	0.33	0.64	0.53	0.67	0.76	0.85	2.90	1.84	0.65	0.46
	Silt, in acre-feet	424	728	1 095	4 360	5 896	10 460	8 267	3 922	21 930	13 230	2 750	1 362
1926	Discharge, in thousands of acre-feet	356	288	446	1 410	2 776	3 560	1 417	536	260	438	255	459
	Silt, percentage	0.30	0.26	0.50	1.00	0.83	0.65	0.83	0.53	0.50	2.09	0.30	0.55
	Silt, in acre-feet	667	468	1 393	8 817	14 390	14 460	7 350	1 775	813	5 724	478	1 578
1927	Discharge, in thousands of acre-feet	280	1 070	719	959	2 990	3 450	2 670	913	1 800	1 050	732	462
	Silt, percentage	0.25	1.27	0.77	0.85	0.81	0.63	0.94	1.08	3.04	1.08	0.62	0.34
	Silt, in acre-feet	437	8 490	3 460	5 095	15 130	13 580	15 690	6 165	34 200	2 089	2 837	982
Average, 9 years	Discharge, in thousands of acre-feet	458	678	692	1 139	2 684	4 434	2 024	960	792	570	540	545
	Silt, percentage	0.66	1.03	0.91	0.95	0.80	0.61	0.84	1.58	2.40	1.21	0.63	0.71
	Silt, in acre-feet	1 879	4 317	3 955	6 729	13 498	13 447	10 642	9 493	11 875	4 312	2 124	2 434

porary discharge through the Laguna Dam sluice-gates is large in relation to the entire flow—as at low stages—the picture of silt conditions may not be dependable. At high stages this disturbing cause is negligible.

TABLE 8.— SUMMARY OF SILT CARRIED IN SUSPENSION BY COLORADO RIVER AT YUMA, ARIZONA.

Year.	Discharge, in acre-feet.	Silt in Suspension.	
		Percentage by weight.	At 100 lb. per cu. ft., in acre-feet.
1911	17 840 000	1.13	126 120
1912	18 358 000	0.65	74 561
1913	11 772 000	0.74	54 798
1914	20 656 000	0.93	119 440
1915	14 645 000	1.16	106 352
1916	22 939 000	1.42	204 006
1917	20 610 000	0.59	76 385
1918	13 146 000	0.63	51 487
1919	10 747 000	1.00	66 832
1920	21 444 000	0.86	114 553
1921	19 428 000	0.86	104 877
1922	17 014 000	0.82	86 970
1923	17 849 000	1.04	115 570
1924	11 424 000	0.83	59 097
1925	12 449 000	0.96	74 418
1926	12 201 000	0.76	57 913
1927	17 095 000	1.06	113 155
Means	16 450 000	0.92	94 500

Classification of Findings.—In 1916, after seven years of observations in more than 1 100 verticals (three samples each), studies of the results of these observations were made by the writer to confirm the conclusion that the admission of river water to the Imperial Canal over flash-boards, at a proposed new head-gate, would reduce the entrained bed load of silt, and, perhaps, in some measure, also, the suspended load, when compared with flow under gates of the ordinary type. The sampling at Yuma has been continued to the present by the U. S. Bureau of Reclamation and, therefore, it has been possible to supplement and extend the studies of 1916.

The observations from 1909 to 1916 were first separated into groups, according as the silt content (by weight) was: (1), Less than 1%; (2) between 1% and 2%; and (3) exceeded 2 per cent. There were 769 verticals in Group (1); 315 in Group (2); and 81 in Group (3).

A combination of the results in all verticals (1 165 in number), with sole regard to the relative quantity of material in suspension near the water surface, at mid-depth, and near the bottom, was then made. Similar combinations were made, too, for each calendar year.

The results of the studies undertaken in 1916 are shown in Table 9. The results of the more recent studies covering the period from 1919 to 1927, are given in Table 10. These data show that a little more material in suspension is to be expected at mid-depth than at the surface and still more near the bottom. In preparing Table 10, the observations were grouped roughly by periods during which the river's silt load remained fairly uniform.

TABLE 9.—DISTRIBUTION OF THE SILT IN SUSPENSION FROM WATER SURFACE TO BOTTOM IN COLORADO RIVER AT YUMA, ARIZONA.

(From observations by U. S. Reclamation Service, 1909 to 1916.)

Year.	Number of verticals.	SILT IN PERCENTAGE BY WEIGHT.		
		At surface.	At mid-depth.	At bottom.
1909	24	0.486	0.594	0.794
1910	66	0.452	0.526	0.571
1911	156	1.040	1.063	1.095
1912	201	0.652	0.660	0.682
1913	195	0.835	0.845	0.889
1914	272	0.974	1.011	1.126
1915	196	0.901	0.922	0.998
1916	55	1.280	1.441	1.485
Weighted average	1 165	0.86	0.89	0.96
Light load	769	0.49	0.51	0.59
Moderate load	315	1.29	1.33	1.40
Heavy load	81	2.67	2.74	2.78

Turbidity Varies with Depth.—It is to be noted that in the Colorado River, which is a very muddy stream, the increase of the suspended load downward from the surface bears no fixed relation to the loads at the surface or at mid-depth. The increase with depth is very nearly constant in the sense of being independent of the total suspended load. Thus, for example, the 769 separate determinations from 1909 to 1916, during periods with a load of about 0.5%, show an actual increase from 0.49% at the surface to 0.59% at the bottom, a percentage difference of 0.10. The 315 determinations in the same years, during periods with a load of 1.0 to 1.5%, showed 1.29% at the surface and 1.40% at the bottom, a percentage increase of 0.11; and the 81 determinations, when the load of silt was heavy, showed 2.67% at the surface and 2.78% at the bottom, or an increase in percentage of 0.11 at full depth. The combination of results in all the 1 165 verticals indicates that the percentage of silt at mid-depth is greater than that at the surface by 0.03 of the weight of water; and that at the bottom the percentage of silt exceeds that at the surface by 0.10. Corresponding comparisons for the period, 1919 to 1927, show the following increases from surface to bottom: For a light silt load, 0.12% of the weight of the water; for a moderate load, 0.08%; and for a heavy load, 0.15 per cent. The combination of results in all the 1 841 verticals of this period show 0.73% silt at the surface; 0.77% at mid-depth; and 0.84% at the bottom, the increase of the percentage being 0.04 from surface to mid-depth, and 0.11 from surface to bottom. In other words, if the percentage by weight of the suspended material in the Colorado River at the surface is represented by p, the percentage of suspended material at mid-depth may be taken at $p + 0.04$ and that at the bottom at $p + 0.10$.

TABLE 10.—DISTRIBUTION OF THE SILT IN SUSPENSION FROM WATER SURFACE TO BOTTOM IN COLORADO RIVER AT YUMA, ARIZONA.

(From observations by U. S. Bureau of Reclamation, 1919 to 1927.)

Year.	Number of days.	Number of verticals.	Average depth, in feet.	SILT IN PERCENTAGE BY WEIGHT.		
				At surface.	At mid-depth.	At bottom.
LIGHT LOAD OF SILT IN SUSPENSION.						
1919	24	44	7.6	0.274	0.274	0.362
1920	51	122	7.3	0.397	0.444	0.502
1921	42	103	8.0	0.316	0.328	0.409
1922	45	60	8.8	0.311	0.311	0.409
1923	38	57	9.3	0.253	0.262	0.346
1924	39	49	8.4	0.248	0.245	0.346
1925	44	85	8.4	0.327	0.415	0.538
1926	57	90	8.0	0.304	0.337	0.466
1927	9	15	6.4	0.170	0.194	0.313
Average weighted	349	625	8.2	0.306	0.332	0.429
MODERATE LOAD OF SILT IN SUSPENSION.						
1919	46	101	9.7	0.486	0.524	0.587
1920	14	29	21.6	0.720	0.670	0.694
1921	40	65	13.0	0.534	0.556	0.593
1922	31	89	14.6	0.634	0.663	0.730
1923	41	110	13.0	0.711	0.705	0.712
1924	46	90	11.0	0.700	0.668	0.733
1925	17	35	15.0	0.598	0.550	0.590
1926	34	82	13.0	0.740	0.778	0.974
1927	71	150	13.0	0.707	0.823	0.883
Average weighted	340	759	13.1	0.603	0.629	0.681
HEAVY LOAD OF SILT IN SUSPENSION.						
1919	27	73	8.5	1.91	1.99	2.07
1920	20	60	11.6	1.42	1.45	1.39
1921	15	43	10.6	2.22	2.34	2.22
1922	27	62	11.3	1.24	1.36	1.55
1923	22	53	9.2	2.36	2.29	2.39
1924	13	37	9.9	1.40	1.44	1.42
1925	18	50	13.1	2.22	2.28	2.21
1926	8	24	7.6	2.31	2.43	2.50
1927	19	55	11.4	1.77	2.02	2.18
Average weighted	169	457	10.4	1.83	1.93	1.96
1919–1927, total and means....	858	1 841	10.6	0.73	0.77	0.84

Quantity of Silt in Relation to River Stage.—The Colorado River at Yuma flows on a bed of very fine sand. The three-point determinations of silt in suspension, of which use was made in the foregoing comparisons, did not include observations at points were the depth was less than 5 ft. The average depths at the included sampling stations are indicated in Table 10. The mean velocity of the river at low stages is generally 2 to 4 ft. At high stages it may go to 6 or 7 ft. per sec. When the river is in equilibrium, there is as much sand being picked up from the bed as is dropped from the suspended load. At such times the river bed, underfoot, appears firm, smooth, and comparatively hard, like a felt-covered floor. With a current of about 3 to 4 ft. per sec., attack on the bed quickly begins whenever there is local acceleration of velocity such as may be caused by even small obstructions, for example, the legs and feet of a man standing in the stream.

Expressed in relation to the weight of the water, there is a wide range in the turbidity of the river, as the long-continued observations disclose. The tabulated results show that the Colorado River is not as muddy at low stages as at moderately high stages—stages with a discharge of 15 000 to 50 000 sec-ft. They also show that the turbidity, expressed in percentages, decreases as the river goes to still higher stages. This does not mean, however, that the river's load of silt decreases with increasing volume. This will become apparent by reference to Fig. 5, which shows the relation of the volume of silt carried in suspension, expressed in acre-feet per month (estimated at 100 lb. per cu. ft.), to the river discharge, this being noted both in acre-feet per month and in second-feet. In plotting Fig. 5, only the mean monthly values as determined for the two periods, 1909 to 1918 and 1919 to 1927 (see Tables 6 and 7), were used. The curve representing volume of silt was first plotted and the percentage curve was deduced therefrom.

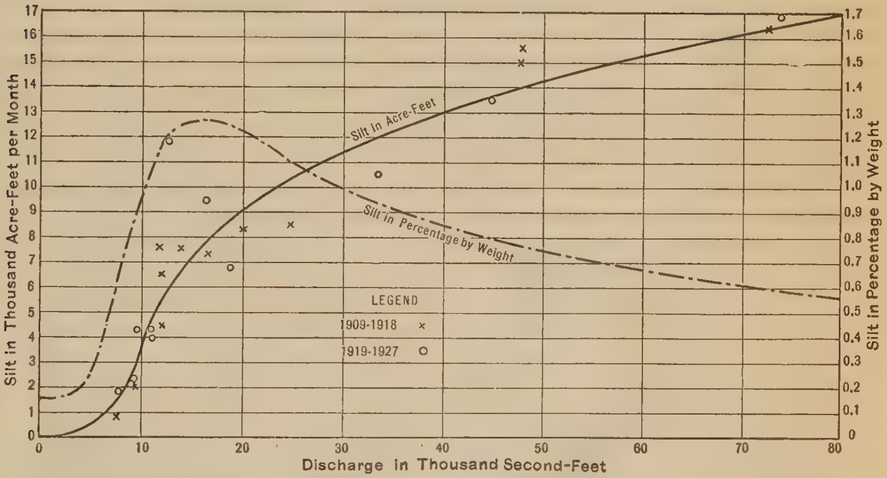

FIG. 5.—RELATION OF AMOUNT OF SILT IN SUSPENSION TO THE DISCHARGE OF THE COLORADO RIVER AT YUMA, ARIZONA.

This study should possibly have been supplemented by a more minute examination of results at the river's lowest stages when the discharge is 6 000 sec-ft., or less, but in view of the complication resulting from the desilting operations at Laguna Dam where upward of 1 000 sec-ft. of desilted water are frequently under diversion into the Yuma Canal, this additional study was not attempted.

The curves in Fig. 5 show a moderate increase of load until the river reaches a 6 000 sec-ft. stage; thereupon the load in suspension, considered in its entirety, rapidly increases from about 1 000 to 10 000 acre-ft. per month, this quantity being reached when the river is at a 25 000 sec-ft. stage. Thereafter, the increase of load continues as the river rises, but the rate of increase is less rapid. At 80 000 sec-ft. the silt carried in suspension amounts to about 16 500 acre-ft. per month, and, for still higher stages, there is an increase of about 700 acre-ft. for each additional 20 000 sec-ft. of discharge. The curve denoting percentage of silt shows that the Colorado River is muddiest when it is discharging 15 000 to 30 000 sec-ft.

The probable volume of silt carried in suspension by the Colorado River changes not only with the discharge of the river, but also from month to month through the year. This change occurs with some regularity, as is made clear by the curves in Fig. 6. In preparing the curves of probable volume the average values ascertained for calendar months for the two periods, 1909-1918 and 1919-1927, were used. Turbidity has two periods of maxima in the year, but with frequent wide departure from mean values. The probable turbidity at any time of the year appears in Fig. 6.

These curves indicate a total annual load of silt in suspension in the Colorado River of about 94 000 acre-ft., estimated at 100 lb. per cu. ft. (which is equivalent to 111 000 acre-ft. at 85 lb. per cu. ft. of deposited silt). This does not include the river's bed load, which, as shown elsewhere, may amount to about 20 000 to 30 000 acre-ft. per annum. Considered from the standpoint of probability and by months, it is to be noted that silt transportation is at a maximum in June at the river's spring high stage. It is at a minimum in the winter months when the river is low. The turbidity is highest in the early spring and in the fall. It is lowest in June at the river's highest stage and in the winter at its lowest stages. The silt carried in suspension since the beginning of the observations by the U. S. Reclamation Service has amounted to 0.90% by weight of the river's total discharge.

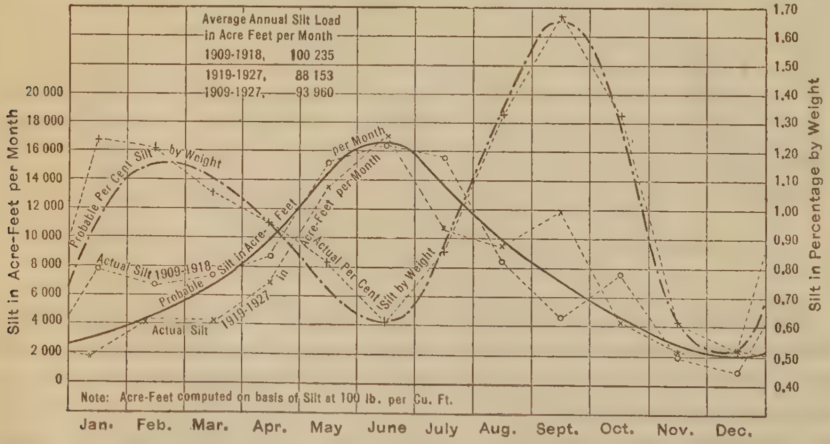

FIG. 6.—SILT IN SUSPENSION IN COLORADO RIVER.

Perhaps there are other causes affecting the degree of muddiness besides the river stage. It is suggested by Mr. J. F. Breazeale that the silt in the lower river in many ways shows "signs of being dispersed, that is, the colloidal particles seem to be carrying negative charges."* He believes that the source of the water and its chemical composition affects the degree of dispersion, or of flocculency, as the case may be, and, therefore, causes a greater or smaller volume of silt in suspension. He explains that, when clays are flocculated, the individual particles are fairly large, but when they are dispersed, carrying charges of negative electricity, each flocculated particle splits into many subdivisions, sometimes so small as to be colloidal.

VARIATION OF SUSPENDED SILT ALONG THE COLORADO RIVER

Since 1925 the U. S. Geological Survey has made determinations of the quantity of material in suspension in the waters of the Colorado River at a number of points up stream from Yuma. Some of the results are accessible in published form.† These data have enabled a preliminary comparison of the quantities of silt in suspension, expressed in percentage of weight, to be made at Lee's Ferry, Grand Canyon, Topock, and Yuma, as shown in Table 11. These values are based, at least in part, on composite samples.

TABLE 11.—SILT IN SUSPENSION AT VARIOUS POINTS OF THE COLORADO RIVER.*
(From U. S. Geological Survey records.)

Period.	PERCENTAGE OF SILT IN SUSPENSION BY WEIGHT.			
	Lee's Ferry.	Grand Canyon.	Topock.	Yuma.
1925:				
August	....	....	....	0.85
August 14-20	....	....	0.79	....
August 22-27	....	....	1.02	....
August 23-29	....	2.54	....	....
August 28, September 3	....	....	2.55	....
September	....	(2.25)	3.12	2.90
October	....	0.86	1.62	1.84
November	....	0.15	0.36	0.65
December	....	0.07	0.19	0.46
1926:				
January	....	0.04	0.16	0.30
January 11	0.06	....	....	....
February	....	0.08	0.13	0.26
February 25	0.10	....	....	....
March	....	0.35	0.27	0.50
March 26	0.63	....	....	....
April	....	1.03	0.81	1.00
May	....	0.61	0.54	0.83
May 24, 26-28, 30	0.77	....	....	....
May 31 to June 6	0.35	....	....	....
June	....	0.26	0.29	0.65
July	....	0.46	0.44	0.83
July 10 and 11	0.35	....	....	....
August	....	0.56	0.54	0.53
September	....	(1.24)	0.81	0.50

*The figures in parenthesis are approximations from incomplete records.

Evidently, in periods, such as October to December, 1925, and January, February, June, and July, 1926, the suspended load increases with the dis-

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* *Technical Bulletin No. 8*, Univ. of Arizona, p. 7.

† *Water Supply Paper No. 596-B*, U. S. Geological Survey.

TABLE 12.—SILT DETERMINATIONS AT SAMPLING STATIONS, IMPERIAL CANAL SYSTEM, SEPTEMBER, 1917.*

Day of month.	Main Canal. near Hanlon Heading.	Main Canal, at Cudahy Check.	Main Canal, at Alamo Mocho.	Main Canal, at Lawrence Heading.	East Side Canal, at Check No. 1.	East Side Canal, at Myrtle Check.	East Side Canal, at Junction Lateral.	Central Main Canal, at 10-ft. drop.	Dahlia Canal, at heading.	Dahlia Canal, at Dahlia Canal No. 12.	Brawley Main Canal, at I. W. Co. No. 4, Del.	West Side Canal, at Wisteria Check.	West Side Canal, at International Boundary.	East Side Canal, at I. W. Co. No. 8, Del.	Tripolium Canal No. 6, at heading.	No. 5 Main Canal, at I. W. Co. No. 5, Del.	Rositas Canal, at heading, Alamo River.	North End Canal, at heading, Alamo River.
1	0.164	0.460	0.431	......	0.426	0.450	0.456	......	0.444	......	0.549	0.308	0.595	0.171	......	0.340	......	0.453
2	0.182	0.627	0.350	......	0.380	0.396	......	......	0.535	0.568	0.357	0.087	0.333	0.388	0.233	0.169	......	0.395
3	0.164	0.350	0.336	......	0.530	0.388	0.227	......	0.380	......	0.444	0.181	0.253	0.230	0.246	0.165	......	0.856
4	0.160	0.287	0.310	......	0.164	0.404	0.348	0.376	0.404	......	0.404	0.079	0.119	......	0.138	0.310	......	0.424
5	0.211	0.236	0.270	......	0.342	0.284	0.258	0.302	0.430	0.462	0.370	0.039	0.049	......	......	0.314	......	0.798
6	0.203	0.289	0.238	......	0.246	0.278	0.224	0.312	0.376	0.282	0.416	0.059	0.289	......	......	0.234	......	0.288
7	0.516	0.357	0.213	......	0.234	0.276	0.304	0.282	0.490	0.646	0.334	0.043	0.057	......	......	0.238	......	0.300
8	0.140	0.200	0.336	......	0.232	0.270	0.212	0.270	0.264	0.514	......	0.306	0.027	......	......	0.218	0.472	0.276
9	0.192	0.225	0.262	0.196	0.216	0.232	0.184	0.250	0.418	0.396	......	0.152	0.093	......	0.176	0.214	0.294	0.724
10	0.138	0.235	0.225	0.242	0.260	0.266	0.230	0.274	0.442	0.406	......	0.198	0.069	......	0.150	0.208	0.266	0.320
11	0.074	0.416	0.250	0.194	0.234	0.360	0.136	0.240	0.372	0.324	......	0.128	0.071	......	......	0.214	0.146	......
12	0.088	0.285	......	0.212	0.216	0.242	0.160	0.238	0.426	0.224	......	0.010	0.053	......	......	0.204	0.162	......
13	0.077	0.235	0.218	0.222	0.202	0.236	0.136	0.156	0.338	0.254	......	0.296	0.083	......	......	0.200	0.240	......
14	0.087	0.231	0.230	0.220	0.174	0.202	0.128	0.158	0.340	0.244	......	0.174	0.139	0.112	......	0.200	0.192	0.180
15	0.074	0.231	0.143	0.188	0.178	0.210	0.128	0.086	0.266	......	......	0.074	0.139	0.174	......	0.268	0.108	0.124
16	0.077	0.151	......	0.206	0.154	0.208	0.120	0.148	0.186	0.188	......	0.076	0.380	0.160	0.114	0.238	0.266	0.332
17	0.144	0.149	0.141	0.150	0.138	0.108	0.092	0.282	0.208	0.260	......	0.078	0.168	0.104	0.108	0.032	0.198	......
18	0.127	0.310	0.111	0.226	0.168	0.120	......	0.240	0.088	0.170	......	0.098	0.086	0.099	.....	0.082	0.070	0.206
19	0.155	......	0.131	......	0.236	0.142	0.088	0.272	0.280	0.154	......	0.082	0.130	0.170	......	0.244	0.224	0.168
20	0.106	......	......	......	0.238	0.200	0.054	0.244	0.360	0.290	......	0.110	0.116	0.124	0.192	0.258	0.254	0.466
21	0.111	0.286	0.093	0.242	0.208	0.410	0.038	0.216	0.288	0.220	......	0.130	0.140	0.196	0.286	0.224	0.308	0.546
22	(0.200)	0.274	0.109	0.212	0.242	0.206	0.152	0.198	0.368	0.198	......	0.112	0.158	0.146	0.290	0.202	0.138	......
23	(0.230)	0.316	0.054	0.232	0.278	0.198	0.142	0.198	0.295	0.180	......	0.072	0.166	0.184	0.274	0.202	0.176	......
24	(0.270)	0.356	0.212	0.238	0.340	0.236	0.112	0.254	0.320	......	......	0.116	0.132	0.150	0.236	0.256	0.260	......
25	(0.300)	0.334	0.274	0.274	0.454	0.224	0.254	0.394	0.340	0.246	......	0.050	0.146	0.172	0.224	0.434	0.220	......
26	(0.600)	0.538	0.248	0.520	0.640	0.304	0.424	0.460	0.434	0.308	0.394	0.200	0.170	0.218	0.240	0.430	......	0.418
27	0.800	0.803	0.268	0.740	0.596	0.432	0.602	0.776	0.462	0.494	......	0.250	0.286	0.234	0.342	0.594	0.628	......
28	(1.000)	0.904	0.494	0.802	0.948	0.808	0.754	0.782	0.922	0.658	......	0.436	0.284	0.320	0.370	0.754	0.896	......
29	(0.600)	1.420	0.717	0.966	0.944	0.790	(1.000)	1.116	1.118	0.722	......	0.536	0.436	0.532	0.504	0.820	0.708	......
30	0.259	0.939	0.876	0.854	1.006	1.202	1.122	0.772	1.226	......	......	0.410	0.418	0.558	0.538	0.588	1.134	......
Average........	[0.145] (0.240)	0.406	0.272	[0.357]	0.358	0.336	[0.262] (0.290)	0.344	0.427	0.351	[0.415]	0.165	0.186	[0.221]	[0.259]	0.295	[0.335]	[0.404]

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*There is a similar sheet for each month, June, 1917, to June, 1918. Figures in parentheses were supplied by the writer. Brackets have been used to show that the reported figures need correction.

tance below the Grand Canyon; in other periods, such as March to May and August, 1926, the suspended load remains fairly constant. In September, 1926, there was even a decrease of suspended load with distance below the Grand Canyon. These results seem to confirm the conclusion that at times the river picks up material from its bed and other times it drops part of its suspended load.

In the Grand Canyon and in the various narrows below, there is coarse sand and gravel in the river bed, and the silt carried by the river is supposed to be in suspension. It was hoped, therefore, that the comparison presented in Fig. 6 might disclose some facts that could be used in estimating the river's bed load, but they give no indication, except possibly that a considerable portion thereof originates below the Grand Canyon.

The Basic Record of Silt Determinations.—The records of the determinations of silt in suspension in the Colorado River, at Yuma, are too voluminous to be incorporated in this paper. The same is true of the records of the sampling results on the Imperial Canal System. Sample sheets illustrate the nature and scope of these records. These are presented as Tables 12 and 13. It is to be noted that the recomputed weighted mean for February, 1920 (Table 13), is 1.70% and the volume of silt carried is 28 000 acre-ft. at 85 lb. per cu. ft., instead of 1.35% and 21 477 acre-ft., as appear in the original record. Similar recomputations for use in this paper were made, where necessary, of all Government records at Yuma since 1919.

TABLE 13.—PERCENTAGE OF SILT IN COLORADO RIVER AT YUMA, ARIZONA, FEBRUARY, 1920.

(Mean percentage of silt, 1.35; discharge, 2 188 316 acre-ft.; silt, 21 477 acre-ft. (at 85 lb. per cu. ft.).)

Day of month.	Distance from left bank, in feet.	Depth, in feet.	Silt in Percentage by Weight.			Discharge, in second-feet.
			Surface.	Mid-depth.	Bottom.	
2	170	8.0	0.47	0.52	0.74	14 200
	350	5.0	0.45	0.61	0.64	
	500	6.0	0.55	0.80	0.69	
6	180	7.0	0.73	0.82	0.91	10 100
	360	4.5	0.68	0.78	0.82	
	510	3.0	0.73	....	0.84	
9	180	7.0	0.59	0.68	0.69	9 800
	360	4.0	0.64	0.66	0.68	
	510	4.0	0.55	0.61	0.66	
13	160	16.0	1.11	1.39	1.49	41 200
	340	12.0	1.91	1.20	1.16	
	550	11.0	1.38	1.22	1.41	
16	170	28.0	1.41	1.30	1.44	31 200
	350	9.0	1.72	1.69	1.34	
	530	7.0	1.11	1.63	1.58	
20	180	14.0	1.29	1.51	1.56	15 900
	330	9.0	1.08	1.30	1.29	
	480	4.5	1.04	1.60	1.62	
23	160	16.0	2.12	2.19	2.66	49 200
	340	12.0	2.45	2.36	1.88	
	550	8.0	2.10	2.64	2.38	
28	150	37.0	1.76	1.91	1.84	65 100
	360	26.0	2.97	1.97	1.74	
	540	12.5	2.40	3.32	2.51	

DISCUSSION

C. S. Howard,* Esq. (by letter).†—This paper makes reference to the quantities of suspended matter found by the U. S. Geological Survey in water samples collected from the Colorado River and reported in *Water Supply Paper No. 596.* The data in that report were from samples collected for analysis of the dissolved mineral matter and are not satisfactory for making comparisons of the loads of suspended matter carried at specific times of the year.

TABLE 14.—Tons of Suspended Matter Carried by the Colorado River. (Based on weighted averages).

Period.	Grand Canyon.	Topock.
October 1, 1925, to September 30, 1926	225 000 000	140 000 000
October 1, 1926, to September 30, 1927	443 000 000	345 000 000
October 1, 1927, to September 30, 1928	189 000 000	209 000 000

The following summary is based on silt samples collected during the past three years (1925-1928), as described in *Water Supply Paper No. 636-B* which gives the complete data. Some of the data are from samples taken at the top, at the bottom, and an "average" sample. Some are from "average" samples alone, which were obtained by opening a bottle near the bottom and drawing it up at such a rate that it was full on reaching the surface. Some data for single days are means of results obtained in several verticals. Others are from only one vertical. Silt samples were collected at least three times a week at each station and more frequently in times of high water. All the results given in Tables 14 to 18, inclusive, are based on samples collected for the determination of suspended matter. A valuable check on the range in

TABLE 15.—Mean Percentages of Suspended Matter in Silt Samples Collected from the Colorado River During the Months of January, April, and June.

Month and year.	Grand Canyon.		Topock.	
	Percentage.	Mean discharge, in second-feet.	Percentage.	Mean discharge, in second-feet.
January, 1926	0.07	6 240	0.25	6 330
April, 1926	2.29	27 900	0.99	27 100
June, 1926	0.74	57 900	0.54	59 400
January, 1927	0.12	6 200	0.17	6 730
April, 1927	1.18	24 100	1.07	22 300
June, 1927	0.77	59 000	0.84	59 100
January, 1928	0.08	7 860	0.51	7 740
April, 1928	0.61	15 200	0.94	16 200
June, 1928	0.65	64 300	1.28	64 500

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† Pub. by permission of the Director, U. S. Geological Survey, Washington, D. C.

quantities of suspended matter and the sudden variations was furnished by determinations of the suspended matter in samples collected daily for analysis of the dissolved mineral matter.

TABLE 16.—MEAN DAILY LOADS OF SUSPENDED MATTER CARRIED BY THE COLORADO RIVER DURING CERTAIN PERIODS FROM NOVEMBER 1, 1925, TO SEPTEMBER 30, 1928.

(Tons, based on the computed quantities for days on which samples were collected during the designated periods.)

Period.	Grand Canyon.	Topock.
November 1, 1925, to February 28, 1926	38 300	70 000
March 1, 1926, to September 30, 1926	971 000	468 000
October 1, 1926, to February 28, 1927	98 100	141 000
March 1, 1927, to September 30, 1927	1 930 000	1 490 000
October 1, 1927, to February 29, 1928	73 500	196 000
March 1, 1928, to September 30, 1928	746 000	843 000

The results in Table 14 indicate that the suspended load carried past the Grand Canyon Station in the period, 1925 to 1927, was greater than the suspended load carried past the Topock Station; but in the period, 1927-1928, the load at Topock was greater. The differences in the loads may be explained by assuming that some of the material is deposited between the two stations and, later, under certain conditions, is scoured out and carried down the river. Since material is transported along the bottom of the river by stream traction, it seems possible that some of the material carried past the Grand Canyon Station as suspended load is carried past the Topock Station as bed load.

TABLE 17.—MINIMUM AND MAXIMUM QUANTITIES OF SUSPENDED MATTER IN THE COLORADO RIVER BETWEEN OCTOBER 1, 1925, AND SEPTEMBER 30, 1928.

Quantity.	GRAND CANYON.			TOPOCK.		
	Percentage.	Date.	Suspended matter, in tons per day.	Percentage.	Date.	Suspended matter, in tons per day.
Minimum	0.03	Dec. 24, 1927	3 720	0.12	Feb. 7, 1927	20 700
Maximum	13.8	Sept. 13, 1927	27 600 000	7.9	Sept. 16, 1927	20 600 000

The variation in the quantities of suspended matter carried past the Grand Canyon and Topock Stations during the year is shown in Tables 15 and 16. Table 15 gives the mean monthly percentage of suspended matter for January, April, and June for each of the two stations. Table 16 gives the mean daily load of suspended matter for certain periods of each year at each station.

The minimum and maximum percentages of suspended matter in the silt samples collected at the Grand Canyon and Topock Stations and the daily loads computed from these percentages are given in Table 17.

These data indicate that for the three-year period the extreme loads (both the largest and smallest) were carried past the Grand Canyon Station. The smaller loads at Grand Canyon, shown in Tables 16 and 17 for the period, October to February, are perhaps due to lack of material to be transported rather than to a deposition above the Grand Canyon Station. This suggestion is based on the fact that for similar discharges during that period, larger quantities are carried past the Topock Station than past the Grand Canyon Station. It has been found that the range in the percentage of suspended matter carried at a given discharge is very large, as shown in Table 18.

TABLE 18.—SUSPENDED MATTER CARRIED BY THE COLORADO RIVER AT GRAND CANYON AT DIFFERENT STAGES.

Date.	Discharge, in second-feet.	SUSPENDED MATTER.	
		Percentage.	Tons per day.
February 9, 1927	6 210	0·08	13 400
October 26, 1926	6 600	0·20	35 600
August 28, 1926	6 280	0.55	93 200
October 4, 1926	6 320	1.36	232 000
December 12, 1927	8 380	0·08	18 100
August 21, 1928	8 840	0.32	76 300
August 19, 1928	8 670	0.57	133 000
October 9, 1926	8 800	0·87	206 000
October 12, 1926	8 630	1·17	272 000

HARRY F. BLANEY,* ASSOC. M. AM. SOC. C. E. (by letter).—This paper is of vital interest now (1929) since it is probable that large sums of money will be spent during the next ten years building structures to control and utilize the waters of the Colorado River. The life of storage reservoirs built will depend upon the quantity of silt transported by the Colorado River.

At various times since 1917 the writer has conducted silt investigations on the Colorado River and in Imperial Valley for the U. S. Department of Agriculture under the general supervision of Samuel Fortier, M. Am. Soc. C. E., and Mr. W. W. McLaughlin.

Under the immediate direction of the late C. E. Tait, M. Am. Soc. C. E., the writer assisted Mr. J. E. Peck in collecting a large percentage of the data shown in Tables 1 and 5. A detailed description of the field and laboratory methods and equipment used is given in a report† published by the U. S. Department of Agriculture.

The results of the mechanical analyses shown in Table 1, indicate that the character of the bed silt as regards size of particles, in the bed of the main canal, varies with the discharge of the Colorado River. Thus, during March, 1917, the mean flow of the river was 9 800 sec-ft., and the sieve analysis for March 14 of that year showed only 0.05% of the recently deposited silt in the bed of the Alamo Canal coarse enough to be retained on a No. 40 sieve. On July 15, 1917, eleven days after the passing of the peak of the spring flood carrying 143 000 sec-ft., the recent deposits in the bed of the Alamo Canal were much coarser in grain. At that time 12.09% was retained on a No. 20 sieve, 20.88% passed the No. 20, but was retained on the No. 40 sieve; 33.4% passed the No. 40 and was retained on the No. 60 sieve; while 7.28% passed a No. 80 sieve. These results indicate that bed silt is transported and that the high waters bring in most of the heavier silt.

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† *Technical Bulletin No. 67*, "Silt in the Colorado River and Its Relation to Irrigation", by Samuel Fortier and Harry F. Blaney, 1928, pp. 13, 36, 37, 38, and 41.

Some appreciable time would be necessary, of course, for the heavier silt to travel down to the lower reaches of the canal system. If this fact is kept in mind and the July 15 analysis of the Hanlon Heading samples is disregarded, no great difference will be apparent between the character of the bed silt near the intake and that in the lower parts of the system.

In computing the volume of suspended silt carried by the Colorado River (see Tables 6, 7, and 8), Mr. Grunsky assumed the dry weight of suspended silt as 100 lb. per cu. ft. Presumably, this is based on the data given in Table 1. The writer* is of the opinion that this value is too high for "suspended silt," because the material in the samples analyzed in Table 1 was mostly sand or "bed silt" and included very little of the finer "suspended silt." Usually, the canal bottom was hard and compact and, at such times, difficult to sample. Published estimates indicate that the weight of 1 cu. ft. of suspended silt ranges from 33 to 93 lb. according to Mr. C. S. Howard.†

That very little of the "suspended silt" was deposited in the beds of the larger Imperial Canals, is indicated by the data shown in Table 5. Mr. Tait, in referring to these data, stated:‡

> "The percentages obtained from the seventeen other stations located on the system below the Hanlon Station show no appreciable reduction, thereby indicating that while great volumes of silt are removed from the canals by mechanical means each year, the amount deposited in the canals is a very small proportion of the total amount at the intake and that most of the silt entering the canals passes on to the lands irrigated."

The results of the mechanical analysis of "suspended silt" samples taken in the Colorado River at Yuma, Ariz., and the Main Imperial Canal at Hanlon Heading, Calif., by Mr. Tait and the writer, indicate that most of the suspended silt was too fine to be retained on a No. 300 sieve. Table 19 gives the results of some typical analyses of suspended silt. At the Yuma Station the total depth was 11 ft., the discharge was 23 500 sec-ft., and the mean river velocity was 4.82 ft. per sec. At Hanlon, the depth to the bottom was 10 ft., the discharge was 5 749 sec-ft., and the mean river velocity was 4.1 ft. per sec.

It will be noted from Table 19 that 73 to 94% of the silt carried in suspension by the Colorado River at Yuma and 77 to 95% in the Main Imperial Canal was fine enough to pass a No. 200 sieve. The difference between the two is small.

A comparison of the data shown in Table 19 with the results in Table 1, indicates that the silt found in the beds of the Imperial Canals is considerably coarser than that carried in suspension at Yuma or Hanlon Heading, since only 1.7 to 34% of the canal bed deposits are finer than a No. 200 sieve.

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* *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, p. 71.

† *Water Supply Paper 636-B*, U. S. Geological Survey, "Suspended Matter in Colorado River in 1925–28", by C. S. Howard, p. 27.

‡ Seventh Biennial Rept., Dept. of Eng., State of California, 1919–1920, p. 118.

TABLE 19.—MECHANICAL ANALYSES OF SUSPENDED SILT AT VARIOUS DEPTHS.

Item No.	Depth, in feet.	Percentage of silt by weight.	PERCENTAGE OF SILT PASSING AND RETAINED ON SIEVES.					
			Passing No. 20, retained on No. 40.	Passing No. 40, retained on No. 60.	Passing No. 60, retained on No. 100.	Passing No. 100, retained on No. 200.	Passing No. 200, retained on No. 300.	Passing No. 300.
NEAR THE MIDDLE OF THE COLORADO RIVER AT YUMA, ARIZ., JULY 27, 1920.								
1	Top	0.310	0	0.26	0.97	4.84	10.13	83.80
2	4.0	0.456	0	1.05	5.94	15.72	17.51	59.78
3	7.0	0.479	0	0.64	4.40	14.37	20.19	60.40
4	10.8	0.614	0	1.09	9.12	16.82	21.22	51.75
IN THE MAIN CANAL AT HANLON, CALIF., 60 FT. FROM THE WEST BANK, JULY 26, 1920.								
5	Top	0.347	0	0.00	1.47	3.25	9.36	85.92
6	4.0	0.363	0	0.37	2.68	4.83	9.13	82.99
7	7.0	0.369	0	0.26	1.90	7.77	9.41	80.66
8	9.8	0.486	0	0.91	5.35	16.34	15.38	62.02

Perhaps the best indication of the weight of 1 cu. ft. of suspended silt (after drying), as carried by the Lower Colorado River, is shown by the data* collected by Mr. C. A. Engle, of the U. S. Indian Irrigation Service, at Parker, Ariz. Water is supplied to the Parker Project by pumping direct from the river into a settling basin. The weight of dry silt contained in 1 cu. ft. of wet deposit taken from the settling basin varied from 42 to 77.7 lb. and averaged 57.5 lb.

Several of the domestic water-works systems of Imperial Valley have settling basins which afford an opportunity to determine the weight and volume of "suspended silt" deposits.† Samples of these deposits taken by Mr. Peck show dry weights of silt per cubic foot of sediment ranging from 32.3 lb. for the softest material to 52.4 lb. for the most compact material, with an average of about 40 lb.

After a recent investigation of the quantity of suspended matter carried by the Colorado River, Mr. Howard stated‡ that suspended matter from daily samples collected for the U. S. Geological Survey, which were allowed to settle in tubes for a period of more than six months, showed a weight equivalent to about 34 lb. per cu. ft. of the deposited material. These samples did not contain the larger particles in their proper proportions, as the daily samples were taken from the bank. Samples of material from the bottom of the river collected in the experiments on the bed load showed a weight equivalent to 98 lb. per cu. ft. of the material after settling for four months.

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* *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, p. 71.

† *Loc. cit.*, p. 70.

‡ *Water Supply Paper 636-B*, U. S. Geological Survey, "Suspended Matter in the Colorado River in 1925–1928", pp. 26–27.

Samples of river deposits* taken by Mr. McLaughlin and the writer along the Colorado River at Laguna Dam, Yuma, and Imperial Valley Heading, indicated that the unit weight of dry silt in 1 cu. ft. of river deposits ranged from 73.2 lb. for the finer material to 90.7 lb. for the coarser, with an average of 81.6 lb. The material was a mixture of bed silt and sediment formerly carried in suspension, taken from the river's edge where it had been deposited at a low stage. Samples of bed silt taken from canals of the Imperial Valley, about the same time, averaged approximately 100 lb. per cu. ft.

Determinations of the silt content of the river water are usually made on a weight basis. A "percentage of silt by weight" is equivalent to the grammes of dry silt contained in 100 grammes of water, and is derived by weighing the water and then the dry silt, and taking the proportion of the latter to the former. The engineer, from a practical standpoint, is more interested in the volume of silt than in its weight. In order to convert the results of silt samplings on a weight basis to volume, it is necessary to assume the weight of dry sediment in 1 cu. ft. of suspended silt.

After giving careful consideration to all available data, the writer and Mr. Fortier came to the conclusion that the average weight of dry silt contained in 1 cu. ft. of suspended silt, as carried by the Lower Colorado River, would be approximately 62½ lb.† The determination of this weight, while only approximate, simplifies the conversion of the silt content of water from a weight to a volume basis in that percentage of silt by weight equals percentage by volume. The writer is of the opinion that this weight will approximate that of suspended silt deposited in the bottom of a reservoir created by a high dam, such as that at Boulder Canyon, provided the deposits are not given an opportunity to dry out and thus destroy their colloidal condition. However, a mixture of suspended and bed silt deposited on the delta of the Colorado River undoubtedly would dry out and compact sufficiently to approach a dry weight of 100 lb. per cu. ft. after a period of years.

Differences in opinion as to the weight of 1 cu. ft. of silt have resulted in estimates of the average volume of suspended silt that would be deposited annually in reservoirs on the Colorado River ranging from 80 000‡ to more than 250 000§ acre-ft. per year.

At the rate indicated by the higher estimate, the proposed Boulder Canyon Reservoir would probably be almost filled with silt in about 100 years, while at the slower rate the deposited silt would occupy about one-third of the reservoir capacity in the same length of time, provided no other dams were built above it during this period.

A report|| published by the U. S. Geological Survey indicates that most of the previous estimates are too low because the annual load of suspended matter computed from samples collected at Grand Canyon was considerably larger than the average annual load computed from samples collected at Yuma.

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* *Technical Bulletin No. 67*, U. S. Dept. of Agriculture, p. 68.

† *Loc. cit.*, pp. 4 and 71.

‡ *Water Supply Paper 395*, U. S. Geological Survey, p. 222.

§ *Transactions*, Am. Soc. C. E., Vol. LXXVI (1913), p. 1480.

|| *Water Supply Paper 636-B*, U. S. Geological Survey, "Suspended Matter in the Colorado River in 1925–1928", by C. S. Howard, p. 28.

This report states that the volume that will be occupied by the suspended matter is unknown, but that it seems probable that estimates based on 33 lb. and 86 lb. per cu. ft. of the deposited material represent fairly well the extreme conditions of deposition. On the basis of 33 lb. per cu. ft., the quantity of suspended matter carried past Grand Canyon in 1925-26 would occupy 313 000 acre-ft. if deposited in a reservoir; the volume for 1926-27 would be 617 000 acre-ft.; and the volume for 1927-28 would be 263 000 acre-ft. On the basis of 86 lb. per cu. ft., the corresponding figures would be 120 000, 237 000, and 101 000 acre-ft., respectively.

E. S. Lindley,* M. Am. Soc. C. E. (by letter).—The observations described in this paper began with an attempt to fix a permanent cross-section; they soon showed its utter instability. Under somewhat similar conditions, the writer found a special application of ordinary rating curves useful.

The conditions referred to are those of the Indus and its tributaries within the Panjab; gradients range from several feet in the mile to less than a foot, in alluvium without any harder formations. Current meters were introduced only during the writer's incumbency, and the observations then available had been made only with surface floats. Supervision had been difficult; observations of higher river stages had generally been impossible; the only good point was that when observations were possible they had been made daily. No individual observation could be accepted as reliable. As a test, as well as for other purposes, it was eminently desirable to try temporary or gradually changing rating curves. In the result, although some of the observations were shown to be utterly unreliable, the majority were very much better than was anticipated.

A most useful tool was found in plotting the square roots of discharges against gauge-levels; this made each rating curve approximately a straight line, and avoided the difficulty of locating a curve of greatly changing curvature.†

The rating curves of these shifting sandy rivers were found to be much more stable than was expected. Often a curve held good from the cessation of monsoon floods until the winter flood season; there were no data for monsoon flood seasons, but smaller floods affected the curves less than had been expected.

While it was constantly necessary to guard against giving these rating curves undue weight against actual observations, opportunity arose of definitely proving them. About six successive observations at one site were quite consistent among themselves, but disagreed with the curve for the season. They were found to be wrong when discharges passing that site were compared with those above and below. When evaluating losses and gains between successive sites, more consistent results were obtained from daily discharges taken from the curves than from actual daily observations.

This relative stability of rating curves can be understood if it be remembered that an individual silt dune across the river will hardly affect water levels above it, although it will produce higher velocities and a steeper gradient in its own area. The gauge at which a given discharge passes depends on the "average" bed level for some distance below the gauge site, that average being reached by weighting divergences from the mean grade according to their nearness to the gauge. (It is not suggested that this average can actually be calculated.) Therefore, a mere shifting of silt banks is likely to have no practical effect on the rating curve, and the relative gauge height at which the curve stands from time to time is some measure of the more extended changes of general bed level.

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† *Minutes of Proceedings*, Inst. C. E., Vol. CCXVII (1923-24), pp. 401-404.

In the observations described in the paper, an attempt was made to establish a dependable value of Kutter's n by applying the average gradient of an 8 000-ft. length to the average cross-section of a 200-ft. length; but this shorter reach may be one of a number in which the depth is either small with high velocity and sharp local gradient, or great with low velocity and flat local gradient; or it may be one in which velocity is either decreasing or increasing. To determine the value of n dependably, it seems necessary to sound a length of channel sufficient to average these conditions. Friction losses and values of n may depend less on the rugosity as ordinarily estimated from the channel surface than on the spacing of decelerating sections from rapids to pools, which may be considered a large-scale rugosity and is independent of the former.

In a paper describing observations in Egypt the author, Mr. Arthur Burton Buckley, asserted* a relation between the value of n and the silt charge carried by the flow. In the discussion a number of possible physical explanations for the existence of such a relation were suggested, none of them really satisfactory. A possible explanation that did not then occur to the writer is that the humps and deeps of large-scale rugosity are likely to be more smoothed out in the floods that carry silt. It would be of interest to know whether the observations described in this paper throw any light on this question.

The author quotes a series of analyses which seem to show that the silt along the entire canal system was similar. In 1915 similar observations were made on the Lower Jhelum Canal in the Panjab, and on some neighboring inundation canals. Samples were collected from just under the surface of silt banks when the canal was not in flow, at a number of points from the head to the tails, outside and inside offtakes. Those of the observations that came under the writer's notice showed fineness increasing regularly with distance from the head. Most offtakes have a tendency to take more than their share of silt, especially the coarser grades; but the pairs of samples at offtakes showed differences insufficient to account for the disappearance of coarser silt; it was not being eliminated by deposition, as there was no steady accumulation of silt anywhere. It was then suggested that even fine silt is subject to attrition, as gravel and shingle are known to be in swifter streams.

The author refers to "boils" on the water surface, and suggests that they are due to the sudden slipping of areas of the river bottom. The writer believes that he has seen them formed over the paved floors above the undersluices of Panjab head-works, where it was improbable that the bed silt had any chance of coming to rest in the course of its flow.

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* "The Influence of Silt on the Velocity of Water Flowing in Open Channels," *Minutes of Proceedings*, Inst. C. E., Vol. CCXVI (1922–23), p. 183.

The author's reference to "rhythmic pulsation as if elongated waves were following each other down the river," recalls some points observed by the writer. At the up-stream pier noses of canal bridges there is generally a rhythmic pulsation of water level, up to about 6 in. at some bridges. The pulsations at adjacent piers are of opposite phase, up at one pier when down on the other. The down-stream "wake" of each pier swings from side to side rather sharply, in time with this pulsation. Although the phenomenon did not seem to have any practical importance, the writer once filled some idle minutes by observing it. A pulsation of the same period was observable at the sides of the canal up stream, diminishing with distance, but still noticeable 2 or 3 miles up stream. The water surface showed slight waves; their approximate spacing divided by the approximate mean velocity of flow roughly agreed with the period of pulsation. It had then become too dark to see whether they were moving; they seemed to be on diagonal lines. No observations of the canal bed were made for comparison, but it has been seen to show dunes 6 to 12 in. high in a 5-ft. depth of water, extending across an appreciable part of the width in single lines, and of about the same wave length as the surface waves; but the dunes would only move very slowly, down stream. Rough figures from memory are a 5-sec. period, 15-ft. wave length, and 3-ft. velocity.

THADDEUS MERRIMAN,* M. AM. SOC. C. E. (by letter).—The author has assembled and referred to much of the available data relating to the suspended load carried by the Colorado, but the important question as to how much material this river transports as bottom load is treated only as an incident. Through the literature the writer has long been familiar with this river, and a recent opportunity of examining it between Black Canyon and Yuma prompts him to submit the following observations and deductions.

As the discharge of the Colorado increases, a substantial part of the increase in wetted cross-section results from a scouring out of its bottom and sides. The high-water marks along the river between Black Canyon and Yuma bear mute yet convincing testimony that this under-deepening, at all points of approximately equal width, where the channel is confined by hard shores, is of practically equal volume.

As the discharge increases in the spring of the year the bed of the stream is lowered and widened and the surface rises until a maximum depth is reached at or about the time of maximum discharge. Gauge heights during the flood season on the Colorado vary much more uniformly than on many other rivers because: (*a*) The source of the water is largely in the melting snows; and (*b*) the regulating effect of the long length of river between Glen and Black Canyons tends to iron out the irregularities of flow. By reason of this approximation to uniformity of discharge, long stretches of the river may, at any one time, be said to be at a proportionately equal flood stage. At such a time, therefore, the Colorado channel within any such stretch will have been excavated to sections corresponding to the river discharge. Now if the rate of discharge be maintained for a period sufficiently long for the water to travel, say, from Black Canyon to Yuma, then all the material excavated between these points will have been carried to and delivered below Yuma. The flow records at the Topock and Yuma gauging stations indicate that this condition frequently occurs.

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* Chf. Engr., Board of Water Supply, City of New York, New York, N. Y.

A flood as it advances picks up material in proportion to the velocity of flow and thus the bottom load is increased, while a receding flood is constantly reducing the burden it carries. The quantity of bed material moved out by any flood is equal to the product of its crest length and the area of the deepening induced within that length. At or near the maximum gauge height the river transports a maximum of bottom load, and as long as the gauge height and the cross-section remain constant the quantity transported is the same for each unit of time.

Every flood coming down the Colorado carries in its bilges sufficient material to refill the space which it excavates. The numerical difference between the volume of material removed from any section of the river bed and the volume later deposited therein is always sensibly equal to zero. Were this not so, the regimen of the river above the deltal area would show progressive changes.

Bearing in mind the foregoing considerations, it is possible to predicate, (1) that at low-water stages the bottom load is small; and (2) that, as the flow varies, the bottom load in any unit of river length will vary directly with the scoured-out area of the wetted cross-section and inversely as the average velocity of flow.

It thus follows that the scoured-out area and the average velocity together constitute a direct measure of the bottom load. That is, the bottom load at any instant in any unit length of the river cross-section is equal to the scoured-out volume of that cross-section divided by the average velocity of flow. Did this equation or condition of equilibrium not obtain, the scoured-out area could never be refilled to its original condition. This fundamental, on which the argument presented is based, may also be stated as follows:

> The bottom load carried per unit of time is equal to the volume of material scoured out per unit of length.

This expression is a simple statement that the continuity of the equilibrium always remains unchanged.

By way of example, if the low-water section area is 3 000 sq. ft. and if this be scoured out by 8 000 additional sq. ft., while the surface rise adds 6 000 sq. ft., then the total section area is 17 000 sq. ft., and the bottom load in any section of unit length is 8 000 cu. ft. divided by the mean velocity, say, 6 ft. per sec. The bottom load thus is 1 333 cu. ft. per unit of channel length, and thus 7.8% of the water volume is solid material in motion. As the flow falls off, the condition of equilibrium being maintained, the bottom load is immediately reduced and the bed is partly refilled. Thus if, with a total section area of 8 000 sq. ft., the scoured-out area is 1 000 sq. ft. and the mean velocity is 5 ft. per sec., then 2.5% of the water volume will be bottom load.

The principles advanced should be of ready application, although the writer is aware that certain practical difficulties will arise. In the first place, the Colorado, even at its lowest stages, carries an appreciable bed load and so the zero datum of the scoured-out area may be determined only through a series of approximations; and, secondly, it may develop, on further analysis, that what the writer has designated as bed load may, in fact, be the total load.

The equation of river equilibrium, herein stated, should also find application to a case such as that which the Colorado below Black Canyon will present after the completion of the reservoir at that place. Here, the existing equilibrium will be destroyed, and the river will immediately set itself to the task of establishing new conditions of stability. The effects produced as this process proceeds may be forecast by means of the principle of the continuity of the equilibrium.

Accurate knowledge of the total silt load carried by the Colorado is of vital importance in connection with the many developments which center about its waters. The volume of suspended load reported by the observations of the various Governmental departments seems too small to account reasonably for the actual erosion from the 155 000 000 horizontal acres of the Colorado water-shed above Yuma. The conditions on this water-shed are such as to facilitate erosion and the forces at work are great and ever-acting, even under the prevailing small rainfall. It is even conceivable that, in times past, under a greater precipitation, the total erosion may not have been markedly greater than it is under the conditions of to-day. A low rainfall in a barren country may easily be the cause of more erosion than several times that rainfall on an area closely covered with vegetation.

The principles presented, when applied to the records of flow and change in cross-section areas at the Yuma, Topock, Bright Angel Creek, and Lee's Ferry gauging stations, will, it is believed, shed much light on the problem of the bed load which the Colorado yearly delivers to its delta. In any event, the indications are that the volume of this load, when determined, will prove to be substantially greater than the suspended load which is now known within reasonable limits.

IVAN E. HOUK,* M. AM. SOC. C. E. (by letter).—The author has contributed much of value to the subject of silt transportation. He has also presented some interesting discussions of miscellaneous river phenomena, such as boils, pulsations, sand waves, etc. The conditions which cause the formation and occurrence of sand waves are fairly well understood, but those resulting in boils and pulsations are more or less uncertain, despite the fact that such phenomena occur in practically all conditions of flowing water.

Sand waves are known to result from the movement of material on the stream bed. Their action can be studied intensively in the larger sluicing works at canal intakes such as have been built on many irrigation and power projects of Western United States. In the settling basin at the Fort Laramie Canal intake, North Platte Irrigation Project, in Wyoming, sand waves frequently form during sluicing operations.* Sometimes they form at the outlet end of the basin and move up stream as fast as a man can walk. (See Figs. 7 and 8.)

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* Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

In 1914 the writer made some investigations of boils and pulsations in the flow of the Miami and Erie Canal at Dayton, Ohio. The section of the canal investigated was about 50 ft. wide and about 3 ft. deep; it had a slope of approximately ½ ft. per mile, and usually carried from 100 to 150 sec-ft. of slightly turbid flow at average velocities of less than 1 ft. per sec. No measurable proportions of sand or silt were transported and no movement of material occurred on the stream bed. The investigations were made in connection with the roughness factor experiments described in the Miami Conservancy District Technical Reports and a detailed description of the location was included in that publication.† However, the results of the investigations of boils and pulsations were not published.

Although the quantities of water flowing in the canal were relatively small, and the velocities relatively low, the conditions of flow were quite irregular. At times, the water surface would be covered with boils of varying magnitude; at other times, it would appear to be perfectly smooth and uniform. Sometimes, the surface would be so smooth that the entire body of water would appear to stop and remain stationary for a few seconds, and then to move on down stream in a solid mass with only minor surface disturbances. On other occasions the current would shift from one side to the other in a very noticeable manner. Although in a few cases the water would seem to boil up from the bottom and spread out on one side, or on all sides, without any apparent regularity, in the majority of cases there was a definite rotary movement. The direction of rotation was practically always contra-clockwise on the left side of the channel, and contra-clockwise in the majority of cases on the right. Although theoretically, this direction of rotation would be caused by the rotation of the earth, it is much more likely that the conditions that caused the formation of the boils also caused the direction of rotation, except for such effects as would be produced by the velocity distribution in the canal. Since the velocities increased from the sides of the channel toward the center, the velocity distribution would tend to cause a contra-clockwise rotation on the left side of the canal.

The investigations of boils and pulsations were made for the purpose of determining what, if any, relation existed between these two phenomena; whether they had any effect on the accuracy of current meter gaugings of discharge; and whether their formation and occurrence were in any way related to the growth of moss on the bed and sides of the channel. Observations consisted of careful, prolonged, current meter measurements of velocity, at two-tenths and eight-tenths of the depths, at certain stations in the cross-section, for the purpose of determining velocity pulsations. Notes were taken regarding size, frequency, location, direction of rotation, and general characteristics of boils. Readings were taken on September 15, October 6 and 9, and November 6, 1914. On the first date, the channel was in ordinary midsummer condition, weeds and grass lining the banks and moss covering the bottom in many places. On the second and third dates, the sides and bottom of the channel had been thoroughly cleaned of weeds, grass, moss, and débris through a distance of about 50 ft. extending up stream from the gauging station; and both banks had been cleaned to some extent for several blocks in both directions. On the last date mentioned, the sides and bottom had been very carefully cleaned of all obstructions for about ¼ mile in each direction.

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* "Sand Control Works at Fort Laramie Canal Intake," by Ivan E. Houk, M. Am. Soc. C. E., *Engineering News-Record*, June 14, 1928, p. 922.

† "Calculation of Flow in Open Channels," by Ivan E. Houk, M. Am. Soc. C. E., Miami Conservancy Dist. Technical Repts., Pt. IV, Dayton, Ohio, 1918, p. 111.

No connection whatever could be observed between the boils noticeable on the surface and the velocity pulsations indicated by the current meter; the variations in velocity at times were greatest when the surface appeared to be entirely free from disturbances, and at other times they were greatest when the surface was covered with boils. The cleaning of the 50-ft. section just above the gauging station seemed to make conditions definitely worse than they were before, especially in regard to pulsations. Cleaning the entire canal for some distance in both directions resulted in better conditions of flow than were observed on the second and third dates. However, the flow was still more irregular than it had been before any cleaning was done. Apparently, the weeds and moss acted somewhat like baffles in a settling basin, breaking up the pulsations instead of creating them as had been thought possible. The cause of the formation and occurrence of the boils and pulsations was not learned. It is quite likely that the pulsations were related in some way to flow conditions at points up stream, such as the discharge from water wheels at plants along the canal which used the flow for power purposes. The boils probably were related in some way to irregularities in the bed of the channel although there was no shifting of materials on the canal bed. Pulsations in velocity were definitely more pronounced at eight-tenths of the depth than at two-tenths, as would naturally be expected. It was concluded that current meter gaugings by the usual two-point method might be in error as much as 4% due to the effect of pulsations.

The author's estimates of quantities carried as bed loads in Imperial Canal and Colorado River, although naturally approximate, are well worth consideration. The estimate of from 25 000 to 30 000 acre-ft. as the annual bed load of the Colorado River at Yuma, Ariz., does not seem unreasonable when studied in connection with the long and careful record of silt samples taken from the river flow at that location. Such an estimate is approximately 25 to 30% of the total average annual silt load indicated by the samples.

The writer does not feel sure that the bed-load problem is as much of a bugaboo as it is usually made out to be; or that a large part, if not all, of the bed load is not properly considered in a carefully designed and regulated system of silt sampling. It is generally assumed that silt sampling determines the suspended load only. However, the writer is inclined to believe that samples taken at, or just above, the river bottom, as approximately one-third of the samples were at Yuma, include a large proportion of bed-load material. Consequently, estimates of annual silt load based on such a record would tend to include a large part of the so-called bed load.

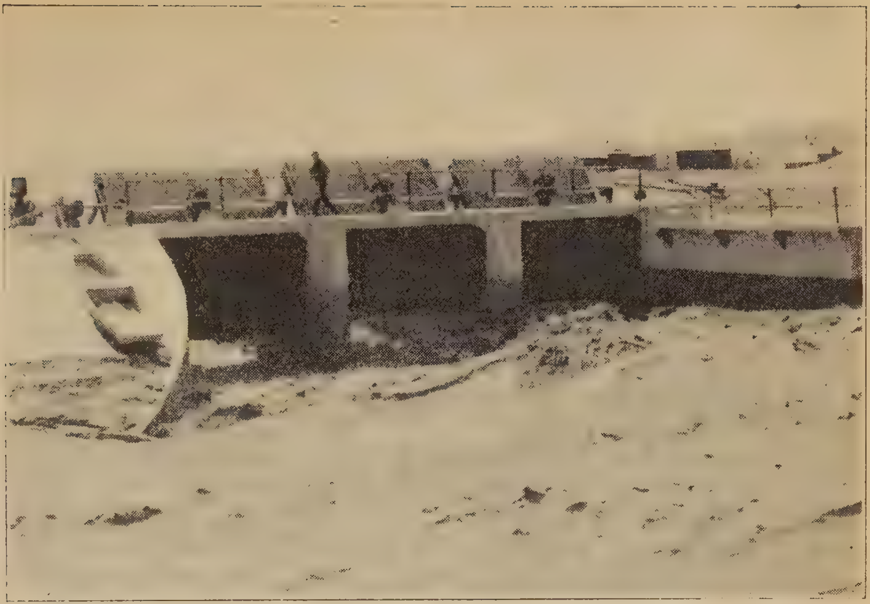

FIG. 7.—VIEW OF TYPICAL SAND WAVES, FORT LARAMIE BASIN, DURING SLUICING PERIOD WITH VELOCITIES VARYING FROM 10 TO 15 FEET PER SECOND.

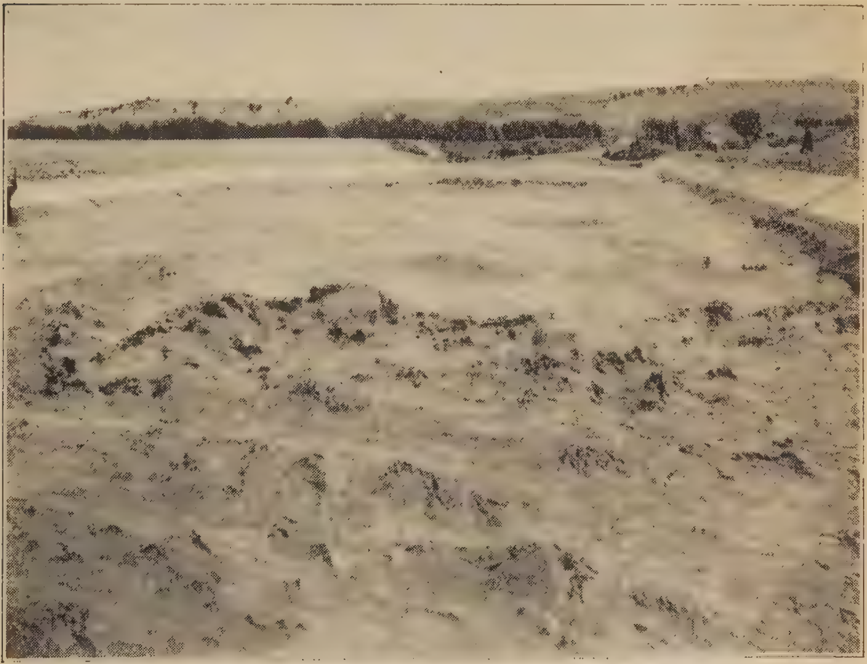

FIG. 8.—SAND WAVES IN SETTLING BASIN AT FORT LARAMIE CANAL INTAKE, DURING SLUICING OPERATIONS.

The writer's idea of bed load in the case of a river like the Lower Colorado is not that it is a solid mass of moving material at the bottom of a channel, into which a silt sampler will refuse to penetrate; but, rather, that it is a comparatively thick mixture of sediment and water of gradually increasing consistency, close to the bed of the stream, which is easily penetrated to appreciable depths by a comparatively heavy sampler and, consequently, is capable of being sampled. Of course, accurate samples of flow at the bottom of the channel could not be expected in the case of extremely large floods in a river like the Colorado; or in the case of large streams transporting heavy loads of gravel, cobbles, or boulders by saltation, unless unusually large sums of money were available for such investigations. However, there undoubtedly are many silt-bearing streams of characteristics such that a carefully regulated and relatively inexpensive system of sampling would determine the total silt load. This would be especially true of artificial channels, such as irrigation canals.

The fact that fairly reliable predictions of reservoir silting can be made from samples of river flow has already been demonstrated. About 1913, the late W. W. Follett, M. Am. Soc. C. E., estimated the average silt load of the Rio Grande at San Marcial, N. Mex., for the period from 1897 to 1912, inclusive, at 19 739 acre-ft.* In 1916, storage of water was begun in Elephant Butte Reservoir, just below San Marcial; and since that time rates of silt deposition within the reservoir have been determined by hydrographic and topographic surveys made at about 5-year intervals. A study of the results shown by the reservoir measurements was made after the 1925 survey and, later, R. F. Walter, M. Am. Soc. C. E., showed that the average annual rate of silt deposition within the Elephant Butte Reservoir, for the period from 1916 to 1925, was 20 508 acre-ft., or only 4% greater than Mr. Follett's estimate.† The latter estimates showed an average silt load equal to 1.66% of the total run-off, whereas the U. S. Bureau of Reclamation's reservoir and stream flow measurements showed an average annual silt deposition equal to 1.64% of the total run-off.

Silt entering the upper end of Elephant Butte Reservoir, although relatively fine, is coarser and heavier than silt in the Colorado River at Yuma. Consequently, the proportion of total silt load carried as bed load probably is greater in the Rio Grande than in the Colorado. The total annual flow in the Rio Grande is much less than in Colorado, floods are much smaller, and bed-cutting during flood periods is much less pronounced.

Mr. Follett's estimate of average annual silt load at San Marcial was based on a 16-year record of stream flow and silt sampling, the first half of which was computed from observations at El Paso, Tex., proper corrections being applied to allow for differences in discharge at the two locations. Only a limited number of samples were available for determining the silt content of the river during the first 8-year period; but a total of 824 was available for the second, which was an average of 103 samples per year for the last 8 years of the 16-year period. Mr. Follett's estimates may have contained compensating errors which would not necessarily exist in other cases. He assumed the average unit weight of dry Rio Grande sediment to be 53 lb. per cu. ft., a figure obtained by securing one 3-in. cube of silt and determining its dry weight. The cube was carefully cut from a sediment bar just above the Mexican Dam, at El Paso, on June 29, 1904, after the spring flood had subsided and after considerable shrinkage had taken place in the silt deposit. The Bureau of Reclamation's subsequent investigations indicate an average weight of 85 lb. per cu. ft. for dry Rio Grande silt. Moreover, it is understood that after Mr. Follett's *Bulletin* was published an error was found in his original computations, and that the correction of this error would have increased his value of 53 lb. per cu. ft.

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* "Silt in the Rio Grande," by W. W. Follett, *Bulletin*, U. S. Dept. of State.

† *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1727.

If it is assumed that Mr. Follett's estimates of silt load volume should have been based on a unit weight of 85 lb. per cu. ft. instead of 53 lb., his estimate of average annual silt load at San Marcial would have been 12 308 acre-ft. instead of 19 739, or only 60% of the Bureau of Reclamation's measured value. Assuming the value obtained from the reservoir surveys to be correct, the estimate made from the samples of river flow, which, in this case, would be mostly suspended sediment, would leave 8 200 acre-ft., or 40% of the true total, as a possible average annual bed load in the Rio Grande. It thus appears that Mr. Follett's use of a relatively low unit weight of silt may have approximately offset his inability to secure accurate quantitative data on bed-load transportation. However, be that as it may, the interesting fact remains that his estimate of average annual silt load was almost identical with the value obtained by careful reservoir surveys.

C. E. GRUNSKY,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).—There is little to add to the discussion of this paper. The question is brought up by Mr. Blaney as to whether 100 lb. per cu. ft. is in fact a proper weight to apply in determining the volume of Colorado River silt from the weight of suspended matter. It is well known that investigators of the character of the silt of the Colorado River, as a rule, have adopted a much smaller figure. However, if the material in the delta region of the Colorado River is examined, all of which was deposited in the course of time by the river, it will be found that 100 lb. is not so very far out of the way. This value was used because it facilitates computation of volume on any other basis of weight and because the few samples taken of the deposit in the stream or canal channels had a weight approximating 100 lb. per cu. ft. The suspended material, freshly deposited, occupies more space and is lighter than it will be after some years, particularly if occasional opportunity is given for it to dry out. How much volume any given weight thereof will occupy in a large reservoir after a series of years has not yet been ascertained.

Some fairly definite basis for estimating a lower limit of the Colorado River's bed load is given in the paper. The gauging work at Yuma has disclosed that at high water the bed of the river may be depressed at that point about as much as the river stage rises above normal. It is not likely that such

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[*] Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.

deepening of channel is coincident at all points of the Lower Colorado, as Mr. Merriman assumes, for the purpose of establishing a basis for a silt formula. If this were the case all the eroded material would appear as the river's suspended load during long periods and there would still remain the difficulty of estimating the bed load—the term, bed load, being used to designate the material rolled along the bottom or moving by saltation within an inch or so of the bottom. It was hoped that the discussion of the paper might disclose other facts than those cited, bearing on bed loads, but this has not been the case.

Mr. Houk is inclined to assume that a part of the bed load of the river is usually reported as suspended material. Whatever the sampling of the water for material in suspension discloses is to be classed as material in suspension no matter how near the bottom the sample is taken. The bed load includes only the material which, under the influence of the current rolls along the bottom up the back of a wave of sand and down its face, or up the long up-stream slope of a gravel or sand-bar and down its steeper down-stream face. There is no movement throughout the mass of submerged sand or gravel. All movement is of the sand grains or gravel particles at the surface of the sand or gravel mass; but this movement carries the individual particles forward over a crest and brings them to rest in valleys or deeps where they remain until the wave or bar has traveled its full length, whereupon any particle in question again travels, by rolling or in minute saltations, the full length of the wave or bar. A gravel bar in the Sacramento River, for example, near Princeton, was about ¼ mile farther down stream at one low-water period than it had been the year before. The entire movement of this bar, which had a length of possibly ½ mile, resulted from the travel of gravel particles by rolling from its up-stream edge over its back and crest to its down-stream end where they would fall to the level of the base of the bar. There they would come to rest as explained and would remain so until all other particles composing the bar had made a similar journey. The bed load of the river at this point is represented by the volume of the bar and its longitudinal displacement in the time interval between observations.

The writer desires to again emphasize the fact that the bed load is in no sense dependent upon, or ascertainable from, the lead of sediment which a river carries in suspension.

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Paper No. 1753

ANALYTICAL SOLUTION OF MASONRY DOMES: UNBALANCED LOADS*

By DAVID C. COYLE,† M. AM. SOC. C. E.

WITH DISCUSSION BY MESSRS. A. FLORIS, AND DAVID C. COYLE

SYNOPSIS

This paper is a continuation of the theory of masonry domes, the elementary formulas of which are given in a paper‡ previously published by the Society. The following discussion deals with the more advanced theory involved in the problems of partial loadings and wind loads.

The assumptions are necessarily in the nature of very rough approximations, but in the investigation of unbalanced loads the aim is not so much to get an exact knowledge of the actual stresses as to obtain a theoretical assurance that in the case of some particular dome these stresses may safely be neglected. The formulas based on roughly approximate assumptions will give results which indicate the order of magnitude of the stresses in question and, in this way, serve as a guide to the judgment in unusual or extreme cases in which previous experience cannot be called upon for guidance.

Following a summary of the notation and the elementary formulas from the previous paper, the discussion deals with four cases: (1) Live load on one-half the dome; (2) live load on the center part only; (3) live load on the haunch only; and (4) wind load.

NOTATION§

The following is the general notation used:

A = meridional stress, in pounds per foot of horizontal section, for dead or balanced loads.

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* Published in August, 1929, *Proceedings*.

† Cons. Engr., New York, N. Y.

‡ "Analytical Solution of Masonry Domes," by David C. Coyle, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 102.

§ *Loc. cit.*, p. 103.

A_L, A_W, A_T = same meridional stress for unbalanced live load, for wind, and for total loads.
B = ring stress, in pounds per foot of profile.
B_L, B_W, and B_T are analogous to A_L, etc.
C = tension in the top ring of a vertical support, or compression in the bottom ring of a vertical lantern, in pounds.
C_L, C_W, and C_T are analogous to A_L, etc.
V_L, V_W, and V_T = shear on the horizontal or vertical planes at any point, due to unbalanced live load, wind, and total, respectively.
W = sum of all balanced loads above a given level, x.
p = balanced load per square foot, at level, x.
G = sum of all partial live loading above level, x.
g = partial live load per square foot, at x.
F = sum of all wind pressures above level, x.
f = wind load per square foot, at x.
x = distance of any point above reference plane.
y = abscissa from axis of dome (plus to the right).
z = horizontal ordinate perpendicular to y (see Fig. 1).
R = radius of curvature of profile at x.
r = horizontal distance from x to axis of dome.
θ = angle between radius, R, and horizontal.
H = radial bursting force on a circle of radius, r, for an area of 1 ft. of circumference times 1 ft. of profile, in pounds per square foot.
e = eccentricity of an unbalanced load.

The following notation applies to pyramoidal domes:

R = radius of curvature of the profile on a diagonal at x.
D = semi-diagonal of a horizontal section at x.
d = semi-diameter of a horizontal section at x.
n = number of sides of polygonal horizontal section.
L = side of a polygon on a horizontal section at x.
ϕ = angle between diagonal and side of polygon.
M = bending in a horizontal element at level, x, in foot-pounds per foot of profile.

The following notation applies to spherical domes:

S_0 = height from top of dome to level, x.
p = uniform weight per square foot of shell.
g = uniform live load per square foot on loaded area.
f = uniform wind load per square foot on the entire surface of a dome.
h = height from center of sphere to point, x ($h = x$ in Fig. 1).
V = weight of lantern, in pounds.
w = weight of shell per cubic foot.
t = thickness of shell, $= \frac{p}{w}$.

The following notation applies to conical domes:

a = vertical height of apex of cone above x.
h = vertical height of top of masonry above x.
s = slant height of top of masonry above x.
r_0 = horizontal radius at top of masonry.
p = uniform weight of shell per square foot.
g and f = uniform live loads and wind loads per square foot.

Elementary Formulas for Balanced Loads*

Conoids.—The ring tension at the level, x, equals

$$B = \frac{W}{2\,\pi\,R}\sec^2\theta - p\,r\tan\theta \quad \ldots\ldots (1)^{\dagger}$$

The meridional compression at the level, x, in pounds per horizontal foot of circumference, equals:

$$A = \frac{-W}{2\,\pi\,r}\sec\theta \quad \ldots\ldots (2)$$

The tension in the top of a cylindrical support, or the bottom of a cylindrical load, applied at the level, x, equals

$$C = \pm\frac{W}{2\,\pi}\tan\theta \quad \ldots\ldots (3)$$

In a closed dome pointed at the top, the stresses near the top are:

$$B = \frac{p\,r^2}{2\,R\sin\theta}\sec^2\theta - p\,r\tan\theta \quad \ldots\ldots (4)$$

$$A = \frac{-p\,r}{2\sin\theta}\sec\theta \quad \ldots\ldots (5)$$

In a closed dome rounded at the top, the stresses are:

$$A = B = -p\,\frac{R}{2} \quad \ldots\ldots (6)$$

The corresponding equations for pyramoids are:

$$B = \frac{W\,D}{n\,L\,R}\sec^2\theta - p\,D\tan\theta \quad \ldots\ldots (7)$$

$$A = -\frac{W}{n}\sec\theta \quad \ldots\ldots (8)$$

$$C = \pm\frac{W\,D}{n\,L}\tan\theta \quad \ldots\ldots (9)$$

The bending in a horizontal element, 1 ft. high, is,

$$\pm M = \frac{W\,L\,D}{12\,n\,R\,d}\sec^2\theta - p\,\frac{L^2\,D}{12\,d}\tan\theta \quad \ldots\ldots (10)^{\ddagger}$$

The equations corresponding to Equations (1), (2), and (3), for spherical domes, are:

$$B = \frac{R^2}{r^2}\,S_0\,p - p\,h \quad \ldots\ldots (11)$$

$$A = -\frac{R^2}{r^2}\,S_0\,p \quad \ldots\ldots (12)$$

$$C = \frac{R}{r}\,S_0\,p\,h \quad \ldots\ldots (13)^{\S}$$

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[*] Positive sign indicates tension.

[†] *Transactions,* Am. Soc. C. E., Vol. 88 (1925), p. 105, Equations (4), (5), and (6).

[‡] *Loc. cit.,* p. 107, Equation (11), corresponding to this formula, is incorrect.

[§] *Loc. cit.,* Vol. LV (1905), p. 214, Equation (4).

For conical domes the stresses are:

$$B = -\frac{p\,r^2}{a} \quad \text{.......................} (14)$$

$$A = -\frac{p\,s^2}{2\,h}\left(\frac{r + r_0}{r}\right) \quad \text{.......................} (15)^*$$

$$C = \frac{s\,p\,r}{2\,a}(r + r_0) \quad \text{.......................} (16)$$

For a cone closed at the top:

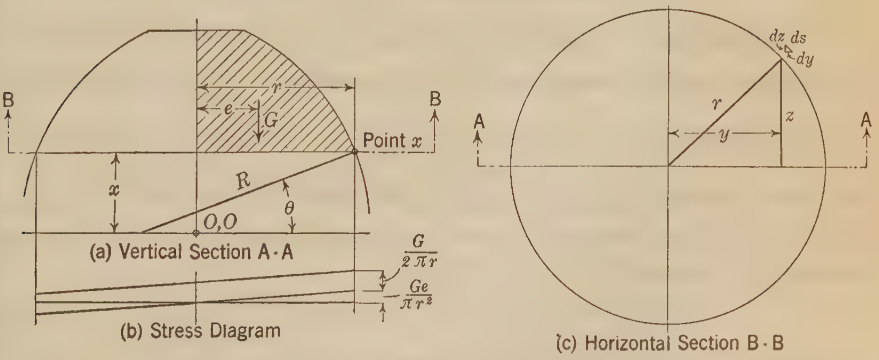
$$B = -\frac{p\,r^2}{a} \quad \text{.......................} (17)$$

$$A = -\frac{p\,s^2}{2\,a} \quad \text{.......................} (18)$$

$$C = \frac{s\,p\,r^2}{2\,a} \quad \text{.......................} (19)$$

CASE 1.—LIVE LOAD ON ONE-HALF OF DOME

Conoidal Formulas.—Assume a homogeneous conoidal dome, with the lines of stress in the center of the thickness of the shell, and with a vertical live load over the right half of its surface (see Fig. 1). The total live load above the level, x, is equal to G, its center of gravity being distant, e, from the axis. The intensity of live load ($= g$ per square foot of surface) is assumed uniform at all points on the same level.



(a) Vertical Section A-A

(b) Stress Diagram

(c) Horizontal Section B-B

FIG. 1.

On the horizontal section at the level, x, it is roughly assumed that the vertical components of the reactions resisting the unbalanced live load are distributed over the section in accordance with the common theory of flexure; that is, the vertical stress equals a uniform stress due to the load plus or minus a bending stress due to the eccentricity, varying uniformly in proportion to its distance from the neutral axis. It is apparent that the bulging of the dome under load will affect the accuracy of this assumption, but not so seriously as to render the results valueless as a guide.

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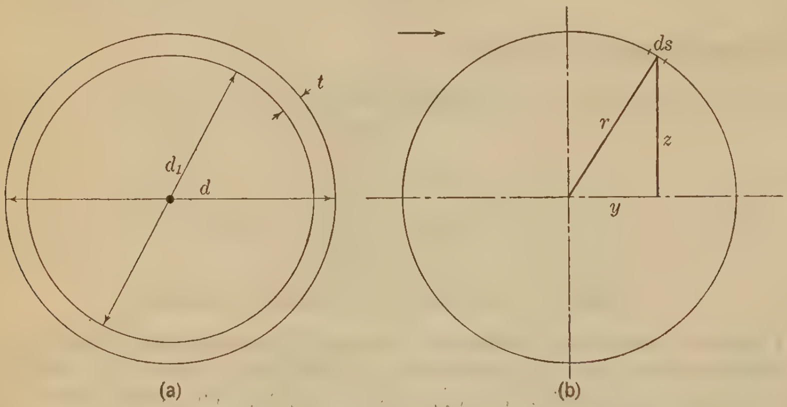
* *Transactions*, Am. Soc. C. E., Vol. LII (1904), p. 303, Equation (73).

Then, the direct vertical load on the horizontal section is $\frac{G}{2\,\pi\,r}$; the moment on the horizontal section is $G\,e$. The section modulus of a cylinder 1 ft. high, about a horizontal axis through the center, is $\pi\,r^2$, derived as follows (see Fig. 2 (*a*)):

$$\text{Section modulus} = \frac{\pi\,(d^4 - d_1^4)}{32\,d} = \frac{\pi}{32\,d}\,(d^2 + d_1^2)\,(d + d_1)\,(d - d_1)$$



(a) (b)

FIG. 2.

Substituting for $d - d_1$, its equivalent, $2\,t$, and if t is small, $d = d_1$, then,

$$\text{Section modulus} = \frac{\pi\,d^2}{4}\,t = \pi\,r^2\,t$$

If d is larger and $t = 1$, the section modulus $= \pi\,r^2$. The extreme fiber stress, in pounds per foot of circumference, due to eccentricity, is $\frac{G\,e}{\pi\,r^2}$. The total vertical load at the point, $y = \pm\,r$, in Fig. 1, is $\frac{G}{2\,\pi\,r}\left(1 \pm \frac{2\,e}{r}\right)$. The vertical load at any point, y, equals $\frac{G}{2\,\pi\,r}\left(1 + \frac{2\,e\,y}{r^2}\right)$. This quantity corresponds with $\frac{W}{2\,\pi\,r}$, the uniform vertical component of the reactions due to balanced loads. A similar derivation, therefore, will give the formulas:

$$B_L = \left(1 + \frac{2\,e\,y}{r^2}\right)\left(\frac{G}{2\,\pi\,R}\sec^2\theta - \frac{g}{2}\,r\tan\theta\right) \ldots\ldots\ldots\ldots (20)$$

$\Big($Note that $\frac{g}{2}$ in Equation (20) corresponds to p in Equation (1), as g covers one-half the section.$\Big)$

$$A_L = -\left(1 + \frac{2\,e\,y}{r^2}\right)\frac{G}{2\,\pi\,r}\sec\theta \ldots\ldots\ldots\ldots\ldots\ldots (21)$$

$$C_L = \pm\left(1 + \frac{2\,e\,y}{r^2}\right)\frac{G}{2\,\pi}\tan\theta \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

In the case of unbalanced loads there is also a shear on the horizontal and vertical sections at any point; that is, the principal stresses are not horizontal and meridional.

Consider the dome to be cut off at the horizontal section, x, and to rest on the top of a thin cylinder. In general, a ring will be required at the top of the cylinder, and below this ring all the stresses in the cylinder are vertical and varying in amount analogous to any eccentrically loaded pier. The stress in the ring is given by Equation (22), and is seen to vary with y, the distance from the neutral axis. Below the ring there is no horizontal shear, as all stresses are vertical. The change in stress in the ring, therefore, must be brought about by tangential shear between ring and dome, which is the same as the horizontal shear in the dome at this section. The dome is assumed to be thin and to carry no radial shear.

The tangential horizontal shear at any point in the level, x, is then equal to $\frac{dC}{ds}$, s being the arc (see Fig. 1(c)). Then, since $\frac{dC}{dy} = \frac{Ge}{\pi r^2} \tan \theta$ and $\frac{dy}{ds} = \frac{z}{r}$,

$$\frac{dC}{ds} = \frac{dc}{dy}\frac{dy}{ds} = V_L = \frac{Gez}{\pi r^3} \tan \theta \ldots\ldots\ldots\ldots\ldots\ldots(23)$$

Special values of Equations (20) to (23) are given in Table 1.

By comparing the formulas for $y = + r$ with those for balanced load, it is evident that in general the maximum values of B and A will be about the same for half loads as they are for the dome fully loaded, since e is about equal to $\frac{r}{2}$. The shear alone is greater with unbalanced load. The values of B_L and A_L are of use in estimating the possible distortion, from which to judge whether the change in curvature is sufficiently great to be dangerous.

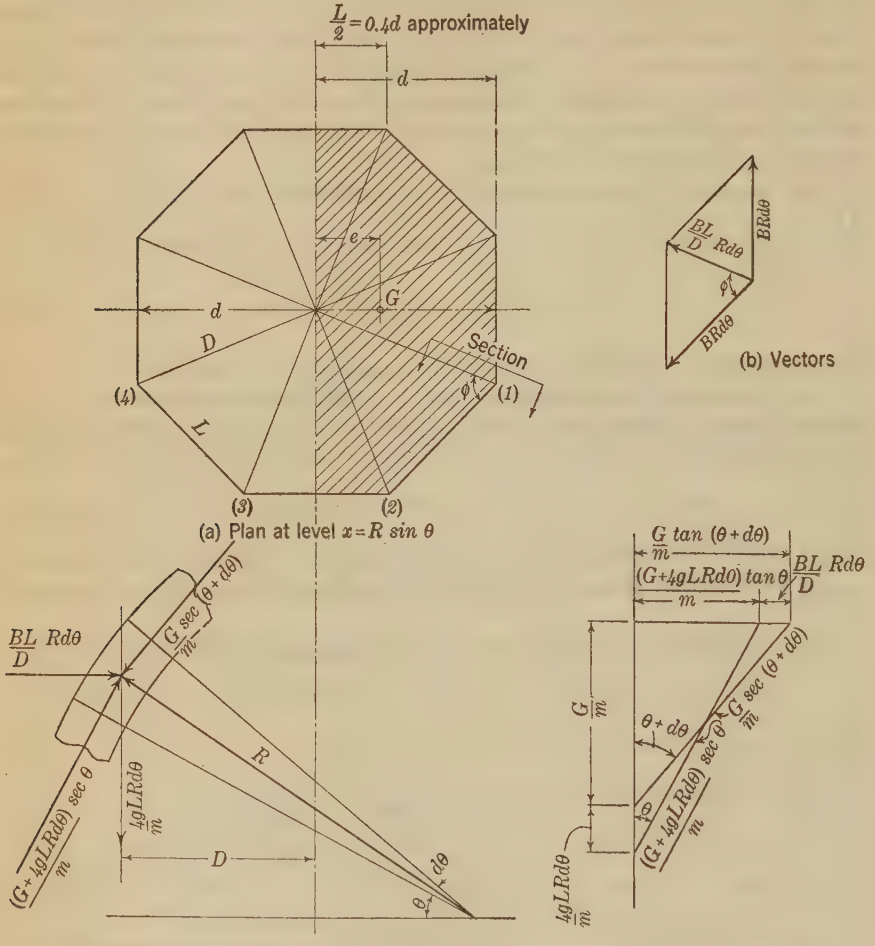
TABLE 1.—SPECIAL VALUES OF EQUATIONS (20) TO (23).

y.	B_L.	A_L.	C_L.	V_L.
$-r$	$\left(1 - \frac{2e}{r}\right)\left(\frac{G}{2\pi R}\sec^2\theta - \frac{gr}{2}\tan\theta\right)$	$-\left(1 - \frac{2e}{r}\right)\frac{G}{2\pi r}\sec\theta$	$\pm\left(1 - \frac{2e}{r}\right)\frac{G}{2\pi}\tan\theta$	0
0	$\frac{G}{2\pi r}\sec^2\theta - \frac{gr}{2}\tan\theta$	$-\frac{G}{2\pi r}\sec\theta$	$\pm\frac{G}{2\pi}\tan\theta$	$\frac{Ge}{\pi r^2}\tan\theta$
$+r$	$\left(1 + \frac{2e}{r}\right)\left(\frac{G}{2\pi R}\sec^2\theta - \frac{gr}{2}\tan\theta\right)$	$-\left(1 + \frac{2e}{r}\right)\frac{G}{2\pi r}\sec\theta$	$\pm\left(1 + \frac{2e}{r}\right)\frac{G}{2\pi}\tan\theta$	0

Pyramoidal Formulas.—The general formulas for a pyramoidal dome would involve an expression for the section modulus of the polygon about a horizontal axis, which is impracticable. The method may be illustrated by using the most common form of pyramoid—the octagonal; any other form may be solved by a similar process.

Let one-half the dome be loaded, as shown in Fig. 3 (a), the load being G with an eccentricity, e. The value of e may be estimated roughly by eye;

the result will be at least as accurate as the distribution of a chance snow load can ever be. The uniform vertical load on each rib due to G equals $\frac{G}{8}$. The moment equals $G\,e$.


$\frac{L}{2}=0.4d$ approximately
d
e
G
d
D
Section
ϕ
(1)
(2)
(3)
(4)
L
$BR d\theta$
$\frac{BL}{D}\,R d\theta$
ϕ
$BR d\theta$
$\frac{BL}{D}\,R d\theta$
$\frac{G}{m}\sec(\theta+d\theta)$
R
$\frac{4gLRd\theta}{m}$
D
$\frac{(G+4gLRd\theta)}{m}\sec\theta$
$d\theta$
θ
$\frac{G}{m}\tan(\theta+d\theta)$
$\frac{(G+4gLRd\theta)}{m}\tan\theta$
$\frac{BL}{D}\,R d\theta$
$\frac{G}{m}$
$\theta+d\theta$
$\frac{G}{m}\sec(\theta+d\theta)$
θ
$\frac{(G+4gLRd\theta)}{m}\sec\theta$
$\frac{4gLRd\theta}{m}$

(a) Plan at level $x=R\sin\theta$

(b) Vectors

(c) Vertical Section on Diagonal at point (1)

(d) Stress Diagrams

FIG. 3.

Let the vertical loads due to moment be: U_1 and U_2, at Points (1) and (2), respectively; then,

$$4\left(U_1\,d+\frac{U_2\,L}{2}\right)=G\,e \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

Since U_2 is approximately equal to $0.4\;U_1$ and $\frac{L}{2}=0.4\,d$, from Equation (24),

$$U_1=\frac{G\,e}{4.6\;d} \text{ and } U_2=\frac{G\,e}{11.6\;d} \quad \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

Stresses at Point (1).—The total vertical load above Point (1), (Fig. 3 (*a*)) is:

$$\frac{G}{8} + \frac{G\,e}{4.6\,d} = \frac{G}{m} \quad \text{(26)}$$

in which,

$$\frac{1}{m} = \left(\frac{1}{8} + \frac{e}{4.6\,d}\right)$$

The total vertical load on $R\,d\,\theta = 4\,g\,L\,R\,d\,\theta \left(\frac{1}{8} + \frac{e}{4.6\,d}\right)$.

By a derivation similar to that used for the elementary formula* (see Fig. 3):

$$\frac{B\,L}{D}\,R\,d\,\theta = \frac{G}{m}\tan(\theta + d\,\theta) - \frac{(G + 4\,g\,L\,R\,d\,\theta)}{m}\tan\theta \quad \text{(27)}$$

With $d\,\theta = \tan d\,\theta$ approaching 0, this reduces to,

$$B_{L_1} = \left(\frac{1}{8} + \frac{e}{4.6\,d}\right)\left(\frac{G\,D}{L\,R}\sec^2\theta - 4\,g\,D\tan\theta\right) \quad \text{(28)}$$

Similarly:

$$A_{L_1} = -\left(\frac{1}{8} + \frac{e}{4.6\,d}\right) G\sec\theta \quad \text{(29)}$$

$$C_{L_1} = \pm\left(\frac{1}{8} + \frac{e}{4.6\,d}\right)\frac{G\,D}{L}\tan\theta \quad \text{(30)}$$

Stresses at Point (2).—By a similar process the formulas for Point (2) are:

$$B_{L_2} = \left(\frac{1}{8} + \frac{e}{11.6\,d}\right)\left(\frac{G\,D}{L\,R}\sec^2\theta - 4\,g\,D\tan\theta\right) \quad \text{(31)}$$

$$A_{L_2} = -\left(\frac{1}{8} + \frac{e}{11.6\,d}\right) G\sec\theta \quad \text{(32)}$$

$$C_{L_2} = \pm\left(\frac{1}{8} + \frac{e}{11.6\,d}\right)\frac{G\,D}{L}\tan\theta \quad \text{(33)}$$

Stresses at Point (3).—At this point the vertical load due to eccentricity is negative:

$$B_{L_3} = \left(\frac{1}{8} - \frac{e}{11.6\,d}\right)\left(\frac{G\,D}{L\,R}\sec^2\theta - 4\,g\,D\tan\theta\right) \quad \text{(34)}$$

$$A_{L_3} = -\left(\frac{1}{8} - \frac{e}{11.6\,d}\right) G\sec\theta \quad \text{(35)}$$

$$C_{L_3} = \pm\left(\frac{1}{8} - \frac{e}{11.6\,d}\right)\frac{G\,D}{L}\tan\theta \quad \text{(36)}$$

Stresses at Point (4).—

$$B_{L_4} = \left(\frac{1}{8} - \frac{e}{4.6\,d}\right)\left(\frac{G\,D}{L\,R}\sec^2\theta - 4\,g\,D\tan\theta\right) \quad \text{(37)}$$

$$A_{L_4} = -\left(\frac{1}{8} - \frac{e}{4.6\,d}\right) G\sec\theta \quad \text{(38)}$$

$$C_{L_4} = \pm\left(\frac{1}{8} - \frac{e}{4.6\,d}\right)\frac{G\,D}{L}\tan\theta \quad \text{(39)}$$

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* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 107, Equation (7).

Shears.—By a process of reasoning similar to that used in regard to conoidal domes, it follows that the horizontal shear in Face (1) to Face (2) (Fig. 3 (*a*)) is:

$$C_{L_1} - C_{L_2} = V_{L_{12}} = \frac{e}{7.6\,d}\,\frac{G\,D}{L}\tan\theta \qquad (40)$$

Similarly,

$$C_{L_2} - C_{L_3} = V_{L_{23}} = \frac{e}{5.8\,d}\,\frac{G\,D}{L}\tan\theta \qquad (41)$$

and,

$$C_{L_3} - C_{L_4} = V_{L_{34}} = \frac{e}{7.6\,d}\,\frac{G\,D}{L}\tan\theta \qquad (42)$$

In the end sections, $V_{L_{11}}$ and $V_{L_{44}} = 0$.

The theory herein presented applies only to domes of solid masonry. When the space between the ribs is filled with glass there is no material to carry the shears shown in Equations (40) to (42). The ribs in this case have to be thick enough to distribute the unbalanced loads by bending. The line of stress does not remain in the center of the ribs, and the present theory does not apply. Domes of this type may be treated by computing the bending caused by distributing any unbalanced load uniformly over the entire surface, after which the stresses are found for the uniform load and added to those due to bending in the ribs.

When the ribs are filled in with solid masonry there is a theoretical bending in any horizontal element, representing the action necessary to carry the bursting stresses to the ribs where they are held by the tension ring stress.

As in the case of balanced loads, $\pm M$ is approximately equal to $\frac{B\,L^2}{12\,d}$, in foot-pounds per foot of profile. Substituting the several values of B_L from Equations (28), (31), (34), and (37), there are found values of $-M$ at the ribs, those of $+M$ between ribs being estimated by interpolation.

Spherical Domes.—The area of a surface with a depth of S_0 is $2\,\pi\,R\,S_0$. Live load covers one-half this surface at a uniform rate of g lb. per sq. ft., the total $G = \pi\,R\,S_0\,g$; also, $\tan\theta = \frac{h}{r}$ and $\sec\theta = \frac{R}{r}$.

For a sphere, Equation (20) becomes,

$$B_L = \left(1 + \frac{2\,e\,y}{r^2}\right)\left(\frac{R^2}{2\,r^2}\,S_0\,g - \frac{g}{2}\,h\right) \qquad (43)$$

Equation (21) becomes,

$$A_L = -\left(1 + \frac{2\,e\,y}{r^2}\right)\frac{R^2}{2\,r^2}\,S_0\,g \qquad (44)$$

Equation (22) becomes,

$$C_L = \pm\left(1 + \frac{2\,e\,y}{r^2}\right)\frac{R}{2\,r}\,S_0\,g\,h \qquad (45)$$

and Equation (23) becomes,

$$V_L = \frac{R}{r^4} S_0 \, g \, e \, z \quad \text{(46)}$$

The value of e for a complete quarter sphere is $\frac{r}{2}$.

Conical Domes.—The area of surface is $s \pi (r + r_0)$; the live load on one-half this surface is,

$$G = \frac{s \pi g}{2}(r + r_0); \; \frac{1}{R} = 0; \; \tan\theta = \frac{r}{a}; \text{ and } \sec\theta = \frac{s}{h}$$

From Equation (20) reduced:

$$B_L = -\left(1 + \frac{2\,e\,y}{r^2}\right)\frac{g\,r^2}{2\,a} \quad \text{(47)}$$

and,

$$A_L = -\left(1 + \frac{2\,e\,y}{r^2}\right)\frac{g\,s^2}{4\,h}\left(\frac{r + r_0}{r}\right) \quad \text{(48)}$$

$$C_L = \left(1 + \frac{2\,e\,y}{r^2}\right)\frac{s\,g\,r}{4\,a}(r + r_0) \quad \text{(49)}$$

$$V_L = \frac{s\,g\,e\,z\,(r + r_0)}{2\,r^2\,a} \quad \text{(50)}$$

If the cone is closed at the top, $r = r_0$, $a = h$, and Equations (47) to (50) may be simplified correspondingly.

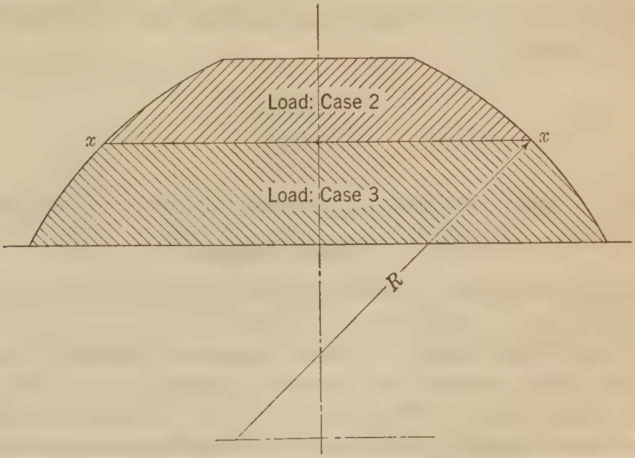


FIG. 4.

CASE 2.—LIVE LOAD IN CENTER OF DOME ONLY

This is shown in Fig. 4, with the load on the area above the level, $x\,x$. Obviously, in the loaded part above $x\,x$, all the stresses are found by the elementary Equations (1) to (25). Below the level, $x\,x$, the formulas are as follows:

$$B_L = \frac{G}{2 \pi R}\sec^2\theta \quad \text{(51)}$$

This value of B may be greater than that for full load, as the negative term of Equation (1) is missing, p being zero.

$$A_L = -\frac{G}{2\,\pi\,r}\sec\theta \ldots\ldots\ldots\ldots\ldots\ldots (52)$$

This is necessarily less than for full load, G being less than W. Therefore, it may be neglected.

$$C_L = \frac{G}{2\,\pi}\tan\theta \ldots\ldots\ldots\ldots\ldots\ldots (53)$$

This may be neglected for the same reason as A in Equation (52).

Case 3.—Live Load on Haunch Only

This is shown in Fig. 4, with the load below the line, $x\,x$. Above the level, $x\,x$, the stresses are as determined by the elementary formulas, for dead load only, and all of them are smaller than the stresses for full load, so they may be disregarded. At the top of the loaded area:

$$B_L = -g\,r\tan\theta \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

This indicates the fact that when B is compression its maximum value is for a load extending up to the level in question, but not above it. Below the level, $x\,x$,

$$B_L = \frac{G}{2\,\pi R}\sec^2\theta - g\,r\tan\theta \ldots\ldots\ldots\ldots\ldots\ldots (55)$$

which is less, in tension or compression, than the values found by Equations (51) or (54), respectively.

$$A_L = -\frac{G}{2\,\pi\,r}\sec\theta \ldots\ldots\ldots\ldots\ldots\ldots (56)$$

$$C_L = \frac{G}{2\,\pi}\tan\theta \ldots\ldots\ldots\ldots\ldots\ldots (57)$$

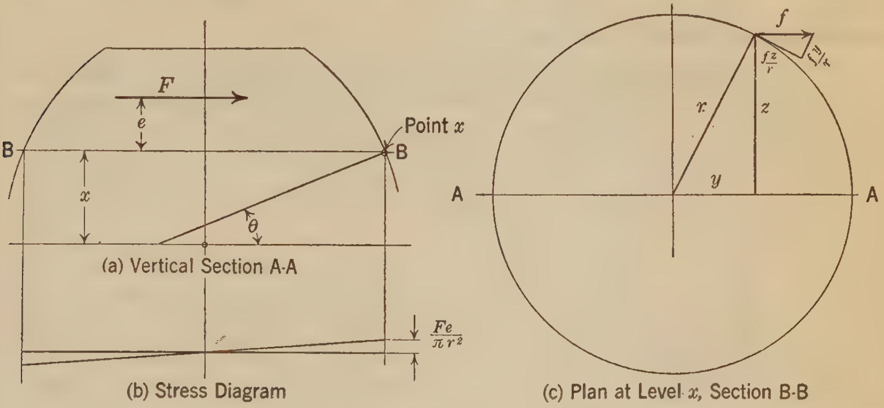
The values of A and C are less than those for full load, and may be neglected.

Case 4.—Wind Stresses

Conoids.—Fig. 5 shows a dome subjected only to a horizontal force, the total of which, above a given section, x, is F. It is assumed for the sake of simplicity, that the effects of wind pressure, friction, and suction are such that every point of the surface is subject to an equal horizontal force, f, per square foot, parallel with the y-axis. The center of pressure is then at the center of gravity of the surface, a distance, e, above the section, x. The wind moment is $F\,e$. The section modulus being $\pi\,r^2$, the extreme fiber stresses are $\pm\frac{F\,e}{\pi\,r^2}$ lb. per ft. of circumference. Similar assumptions are involved as in the case of partial live load.

The vertical component of the stress in the dome at any point in the horizontal section is then $\frac{F\,e\,y}{\pi\,r^3}$. The horizontal force, f, per square foot, parallel to the y-axis, has components: $\frac{f\,y}{r}$, radial, and $\frac{f\,z}{r}$, tangential.

Consider a voussoir (Fig. 6 (*a*)), 1 ft. long and $R\,d\,\theta$ high along the meridion, distance, y, from the neutral axis. The values of r, y, and e are not the same for θ and for $\theta + d\,\theta$, but when $d\,\theta$ approaches zero, the differentials of these values disappear without affecting the result. They are omitted, therefore, in the calculations.



(a) Vertical Section A-A

(b) Stress Diagram

(c) Plan at Level x, Section B-B

FIG. 5.

The forces acting on the voussoir in the radial plane, are as follows:

1.—Meridional thrust from above, $\frac{F\,e\,y}{\pi\,r^3}$ sec $(\theta + d\,\theta)$, of which the horizontal component is, $\frac{F\,e\,y}{\pi\,r^3}$ tan $(\theta + d\,\theta)$, and the vertical component is $\frac{F\,e\,y}{\pi\,r^3}$.

2.—Meridional thrust from below: The total force on the dome down to the bottom level of the voussoir is $F + 2\,\pi\,r\,R\,d\,\theta\,f$. The meridional thrust is $(F + 2\,\pi\,r\,R\,d\,\theta\,f)\,\frac{e\,y}{\pi\,r^3}$ sec θ, of which the horizontal component is $(F + 2\,\pi\,r\,R\,d\,\theta\,f)\,\frac{e\,y}{\pi\,r^3}$ tan θ, and the vertical component is $(F + 2\,\pi\,r\,R\,d\,\theta\,f)\,\frac{e\,y}{\pi\,r^3}$.

3.—The horizontal local wind force, f per sq. ft., on the voussoir equals $f\,R\,d\,\theta$, and the component in a radial plane is $f\,R\,d\,\theta\,\frac{y}{r}$.

4.—The resultant of the vertical components of the ring stress, equal to the difference between the vertical components of Force 1 and of Force 2, or $\frac{2\,R\,d\,\theta\,f\,e\,y}{r^2}$.

5.—The resultant of the horizontal components of the ring stress. The value of the ring stress is $B_W\,R\,d\,\theta$.

Let Resultant 5 be called $H\,R\,d\,\theta$. Then, the sum of all the horizontal forces acting in the radial plane may be set equal to zero, thus:

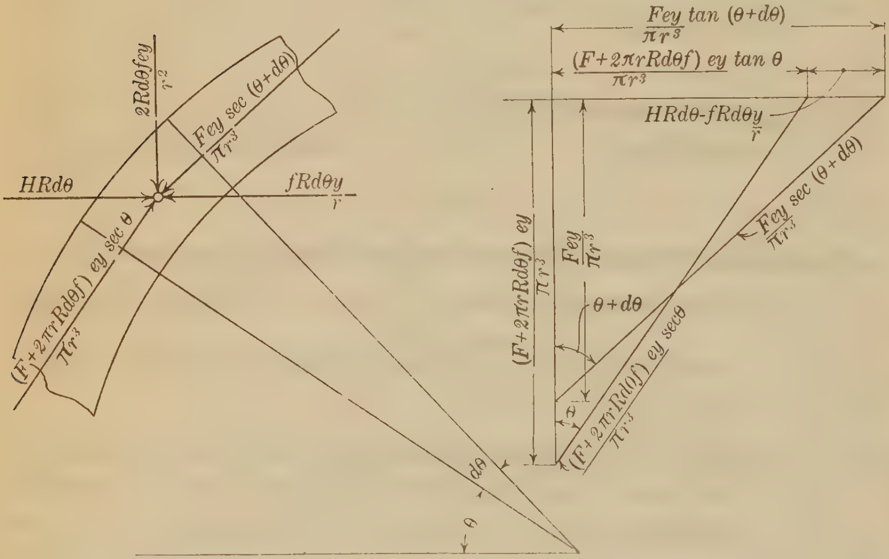
$$H R d\theta + (F + 2\pi r R d\theta f)\frac{e y}{\pi r^3}\tan\theta - \frac{F e y}{\pi r^3}\tan(\theta + d\theta)$$

$$- f R d\theta \frac{y}{r} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

Substituting for $\tan(\theta + d\theta)$ its equivalent, $\frac{\tan\theta + \tan d\theta}{1 - \tan\theta\tan d\theta}$, simplifying, and dividing by $\tan d\theta = d\theta$:

$$H R + \frac{\frac{2 R f e y \tan\theta}{r^2} - \frac{F e y}{\pi r^3}\tan^2\theta - \frac{2 R f e y}{r^2}\tan^2\theta\tan d\theta - \frac{F e y}{\pi r^3}}{1 - \tan\theta\tan d\theta}$$

$$- \frac{f R y}{r} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (59)$$


$\frac{2Rd\theta fey}{r^2}$
$\frac{Fey}{\pi r^3} sec (\theta+d\theta)$
$HRd\theta$
$\frac{fRd\theta y}{r}$
$\frac{(F+2\pi rRd\theta f)}{\pi r^3} ey\ sec\ \theta$
$d\theta$
θ
$\frac{Fey\ tan\ (\theta+d\theta)}{\pi r^3}$
$\frac{(F+2\pi rRd\theta f)\ ey\ tan\ \theta}{\pi r^3}$
$HRd\theta - \frac{fRd\theta y}{r}$
$\frac{Fey}{\pi r^3} sec\ (\theta+d\theta)$
$\frac{(F+2\pi rRd\theta f)\ ey}{\pi r^3}$
$\frac{Fey}{\pi r^3}$
$\theta+d\theta$
θ
$\frac{(F+2\pi rRd\theta f)}{\pi r^3} ey\ sec\theta$

(a) Section of Voussoir (b) Stress Diagram

FIG. 6.

Let $d\theta = \tan d\theta = 0$; then:

$$H = \frac{F e y}{\pi R r^3}(1 + \tan^2\theta) - \frac{2 f e y}{r^2}\tan\theta + \frac{f y}{r} \ldots\ldots\ldots (60)$$

The horizontal ring stress, B_W, is equal to $H r$:

$$B_W = \frac{F e y}{\pi R r^2}(\sec^2\theta) - \frac{2 f e y}{r}\tan\theta + f y \ldots\ldots\ldots (61)$$

From Fig. 6 it is evident that the meridional thrust is:

$$A_W = -\frac{F e y}{\pi r^3}\sec\theta \ldots\ldots\ldots\ldots\ldots\ldots (62)$$

Consider the dome to be supported on a thin cylinder, not capable of carrying any radial shear. The wind will be resisted by tangential shears in the cylinder. A ring will be required at the top of the cylinder. The tension in this ring is the horizontal component of the meridional stress, multiplied by r:

$$C_W = \frac{F\,e\,y}{\pi\,r^2}\tan\theta \qquad (63)$$

The total shear applied by the dome to the top of the ring at any point is the sum of (1) the tangential shear applied through the ring to the cylinder; and (2) the shear required to produce the change in ring tension occurring at this point. The latter will be analyzed first. Consider an element, $d\,s$, of this tension ring. The change in tension in the elemental distance, $d\,s$, is caused by shear applied by the dome. Then, since $\frac{d\,C}{d\,y} = \frac{F\,e}{\pi\,r^2}\tan\theta$ and $\frac{d\,y}{d\,s} = \frac{z}{r}$ (see Fig. 1):

$$\frac{d\,C}{d\,s} = \frac{d\,C}{d\,y}\,\frac{d\,y}{d\,s} = \frac{F\,e\,z}{\pi\,r^3}\tan\theta \qquad (64)$$

The direct shear due to wind, carried by the cylinder, is a total of F, and varies in intensity from a maximum at $y = 0$ to zero at $y = \pm\,r$. Let the maximum at $y = 0$ be $\frac{F}{\pi\,r}$, which is derived as follows (Fig. 2 (*b*)): Let the shear be proportional to z. At $d\,s$ the shear equals $\frac{a\,z}{r}\,d\,s$, in which, $\frac{a}{r}$ is a constant. The component parallel to the y-axis is $\frac{a\,z^2}{r^2}\,d\,s$; but $d\,s = d\,y\,\frac{r}{z}$. Therefore, the component $= \frac{a\,z}{r}\,d\,y$, and the total, $F = \frac{a}{r}\int_{-r}^{+r} z\,d\,y$. Since $\int_{-r}^{+r} z\,d\,y$ $=$ area of the circle $= \pi\,r^2$; $F = \pi\,r\,a$, or $a = \frac{F}{\pi\,r}$. Therefore, the shear $= \frac{F\,z}{\pi\,r^2}$ per foot at any point and varies from $\frac{F}{\pi\,r}$ at $z = r$ to 0 at $z = 0$. Then, the shear at any point will be about $\frac{F}{\pi\,r}\,\frac{z}{r} = \frac{F\,z}{\pi\,r^2}$, of which the component parallel to the wind is $\frac{F\,z^2}{\pi\,r^3}$. The sum of these components is,

$$\int_{-r}^{+r}\frac{F\,z^2}{\pi\,r^3}\,d\,s = \int_{-r}^{+r}\frac{F\,z}{\pi\,r^2}\,d\,y$$

Now, $\int_{-r}^{+r} z\,d\,y$ is the area of the circle $= \pi\,r^2$; so the sum of components parallel to F is $\frac{F}{\pi\,r^2} \times \pi\,r^2 = F$, as required.

The ring tension is greatest at the leeward side, so the tendency of the shear between the dome and the ring is, in a sense, to drag the ring toward the left. The tendency of the shear between the dome and the ring due to the direct wind thrust is, in a sense, to drag the ring toward the right. Therefore, the total horizontal shear at any point is the difference of these two shears, or:

$$V_W = \frac{F\,e\,z}{\pi\,r^3}\tan\theta - \frac{F\,z}{\pi\,r^2}\ldots\ldots\ldots\ldots(65)$$

At the neutral axis, where $z = r$, this shear is a maximum of $\frac{F\,e}{\pi\,r^2}\tan\theta - \frac{F}{\pi\,r}$. At the ends of the y-axis, the shear is zero since z is 0.

Pyramoids.—As in the case of unbalanced live load, the method will be illustrated with the octagonal pyramoid shown in Fig. 3. Consider it to be acted upon by a uniform horizontal force of f per sq. ft. over its entire surface, making a total shear, F, with an eccentricity, e.

The wind moment is $F\,e$, and the vertical loads due to this moment on the ribs are, as before,

$$U_1 = \pm\frac{F\,e}{4.6\,d}, \text{ and } U_2 = \pm\frac{F\,e}{11.6\,d}$$

Stresses at Point (1).—The forces acting on a voussoir of this rib, in the plane of the diagonal, are as follows:

(*a*) Meridional thrust from above, $\frac{F\,e}{4.6\,d}\sec(\theta + d\,\theta)$.

(*b*) Meridional stress from below, $(F + 8\,f\,L\,R\,d\,\theta)\frac{e}{4.6\,d}\sec\theta$.

(*c*) Horizontal wind on voussoir, component in diagonal plane,

$$f\,L\sin\phi\,R\,d\,\theta = \frac{d}{D}f\,L\,R\,d\,\theta.$$

(*d*) Horizontal resultant of ring stress in diagonal plane, $\frac{B\,L}{D}R\,d\,\theta$.

Equating the sum of all horizontal components, *a*, *b*, *c*, *d*, to 0, and solving in the usual manner:

$$B_{W1} = \frac{e}{4.6\,d}\left(\frac{F\,D}{L\,R}\sec^2\theta - 8\,f\,D\tan\theta\right) + f\,d\ldots\ldots\ldots(66)$$

Similarly,

$$A_{W1} = -\frac{e}{4.6\,d}F\sec\theta\ldots\ldots\ldots\ldots(67)$$

$$C_{W1} = \frac{1}{2}\,\frac{F\,e}{4.6\,d}\tan\theta\sec\phi;\ \sec\phi = \frac{2\,D}{L}\ldots\ldots\ldots(68)$$

$$C_{W1} = \pm\frac{e}{4.6\,d}\,\frac{F\,D}{L}\tan\theta\ldots\ldots\ldots\ldots(69)$$

Stresses at Point (2).—By analogy:

$$B_{W2} = \frac{e}{11.6\,d}\left(\frac{F\,D}{L\,R}\sec^2\theta - 8\,f\,D\tan\theta\right) + \frac{f\,L}{2}\ldots\ldots\ldots(70)$$

$$A_{W2} = -\frac{e}{11.6\,d}F\sec\theta\ldots\ldots\ldots\ldots(71)$$

$$C_{W2} = \pm\frac{e}{11.6\,d}\,\frac{F\,D}{L}\tan\theta\ldots\ldots\ldots\ldots(72)$$

Stresses at Point (3):

$$B_{W3} = -\frac{e}{11.6\,d}\left(\frac{F\,D}{L\,R}\sec^2\theta - 8\,f\,D\tan\theta\right) - \frac{f\,L}{2} \quad \text{.........} \quad (73)$$

$$A_{W3} = +\frac{e}{11.6\,d}\,F\sec\theta \quad \text{....................} \quad (74)$$

$$C_{W3} = \mp\frac{e}{11.6\,d}\,\frac{F\,D}{L}\tan\theta \quad \text{..................} \quad (75)$$

Stresses at Point (4):

$$B_{W4} = -\frac{e}{4.6\,d}\left(\frac{F\,D}{L\,R}\sec^2\theta - 8\,f\,D\tan\theta\right) - f\,d \quad \text{.........} \quad (76)$$

$$A_{W4} = +\frac{e}{4.6\,d}\,F\sec\theta \quad \text{....................} \quad (77)$$

$$C_{W4} = \mp\frac{e}{11.6\,d}\,\frac{F\,D}{L}\tan\theta \quad \text{..................} \quad (78)$$

Shears include the change in C_W minus the direct wind shear, as follows:

$$V_{W12} = \frac{e}{7.6\,d}\,\frac{F\,D}{L}\tan\theta - 0.15\,F, \text{ approximately} \quad \text{........} \quad (79)$$

$$V_{W23} = \frac{e}{5.8\,d}\,\frac{F\,D}{L}\tan\theta - 0.21\,F, \text{ approximately} \quad \text{..........} \quad (80)$$

The moment in a horizontal element of the face equals $\pm\,M$ (approximately) $= \frac{B\,L^2}{12\,d}$ due to thrusts. To this is added the moment due to wind directly applied: For Line (1) (1) and Line (4) (4) (Fig. 3), $M = \frac{f\,L^2}{12}$; for Line (1) (2) and Line (3) (4), $M = \frac{f\,L^2}{12\sqrt{2}}$; and for Line (2) (3), $M = 0$.

The stresses due to these moments are to be added to the ring stresses found from Equations (66), etc.

Wind, spherical domes, etc., formulas analogous to Equations (43), etc., can be derived by substituting the factors for the sphere, cone, etc.

Maximum Stresses

It is possible, by the use of the foregoing equations, to obtain for any point in a dome the combined horizontal ring stresses, meridional thrusts, and shears on the horizontal plane, for dead, live, and wind loads. Only in extreme cases, such as unprecedented size or shape, would it be necessary, of course, to go into such detailed investigation. Supposing it to be desirable in a given case, having found the total stresses, B_T, A_T, C_T, and V_T, it remains to find the maximum or principal stresses, and the direction in which they lie. The following method is derived from Rankine.

Consider an elementary surface, $o\,a\,b$ (Fig. 7), with its sides perpendicular to A_T, B_T, and the principal stress, A_P. A_T and B_T are positive as tension in the directions, $A\,o$ and $B\,o$; the balancing stress, A_P, is positive as tension in the direction, $o\,e = o\,f$. The shear, V_T, is positive as clockwise on the vertical plane, and is, therefore, opposite in sign to the direct stresses, A_T and B_T.

Let $d\,o$ and $c\,o$ be the total components acting on the element parallel to the axes; their balancing force is the same as $o\,f$, the normal force, since there is no tangential force in a principal plane.

$$c\,o = A_T\,o\,b - V_T\,o\,a = A_T\,a\,b\cos\alpha - V_T\,a\,b\sin\alpha$$
$$d\,o = B_T\,o\,a - V_T\,o\,b = B_T\,a\,b\sin\alpha - V_T\,a\,b\cos\alpha$$
$$o\,f = d\,o\sin\alpha + c\,o\cos\alpha$$
$$f\,e = c\,o\sin\alpha - d\,o\cos\alpha$$

$$A_P = \frac{o\,f}{a\,b} = B_T\sin^2\alpha - V_T\sin\alpha\cos\alpha + A_T\cos^2\alpha - V_T\sin\alpha\cos\alpha$$
$$= A_T\cos^2\alpha + B_T\sin^2\alpha - 2\,V_T\sin\alpha\cos\alpha \quad \text{.........(81)}$$

$$V_P = O = \frac{f\,e}{a\,b} = A_T\cos\alpha\sin\alpha - V_T\sin^2\alpha - B_T\cos\alpha\sin\alpha$$
$$+ V_T\cos^2\alpha = O \quad \text{........................(82)}$$

from which,

$$\tan 2\,\alpha = \frac{-2\,V_T}{A_T - B_T} \quad \text{........................(83)}$$

A similar demonstration will show that:

$$B_P = A_T\sin^2\alpha + B_T\cos^2\alpha + 2\,V_T\cos\alpha\sin\alpha \quad \text{........(84)}$$

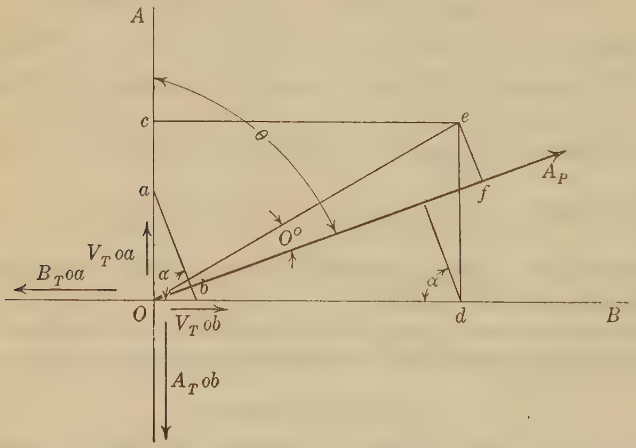


FIG. 7.

EXAMPLES

The foregoing theory may be illustrated by the following examples.

Example 1.—Unbalanced Load.—Consider the dome shown in Fig. 8 having a spherical surface of radius, 200 ft., and $r = 100$ ft.; let the dead load consist of 8-in. concrete weighing 100 lb. per sq. ft., and the live load, 10 lb. per sq. ft. on the right half only.

In Table 2, stresses are given for various values of θ, r, and e. (See Fig. 8.) They were determined by substituting the given data* in Equations (20), (21), and (23).

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* The data given are approximate.

TABLE 2.—STRESSES DUE TO UNBALANCED LIVE LOAD.

y.	B.	A.	V.
$\tan\theta_1 = 1.74$; $\sec\theta_1 = 2$; $r = 100$ ft.; $e = 45$ ft.; $G = 170\,000$ lb.; and area of dome = 34 000 sq. ft.			
+100	− 630	− 1 030	0
0	− 330	− 540	425
−100	− 30	− 50	0
Dead load stress	−6 600	−10 800	0
$\tan\theta_2 = 2.65$; $\sec\theta_2 = 2.82$; $\sec^2\theta_2 = 7.95$; $r = 71$ ft.; $e = 31$ ft.; area of dome above this level = 16 400 sq. ft.; and $G = 80\,000$ lb.			
+ 71	− 800	− 960	0
0	− 430	− 510	420
− 71	− 60	− 70	0
Dead load stress	−8 300	−10 400	0
$\tan\theta_3 = 5.6$; $\sec\theta_3 = 5.7$; $\sec^2\theta_3 = 32.5$; $r = 35$ ft.; $e = 15$ ft.; area = 3 700 sq. ft.; and $G = 18\,500$ lb.			
+ 35	− 910	− 890	0
0	− 490	− 480	400
− 35	− 70	− 70	0
Dead load stress	−9 900	− 9 800	....

It is evident that the stresses due to unbalanced live load are not, in this case, very alarming as compared with those due to a balanced load. The distortion may be roughly computed as follows: Suppose the modulus of elasticity to be 2 000 000 for compression and 1 000 000 for shear.

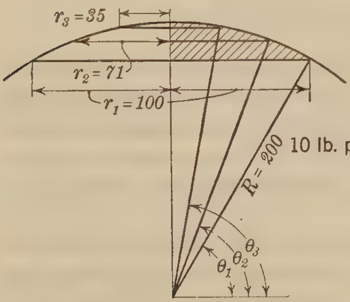

FIG. 8.

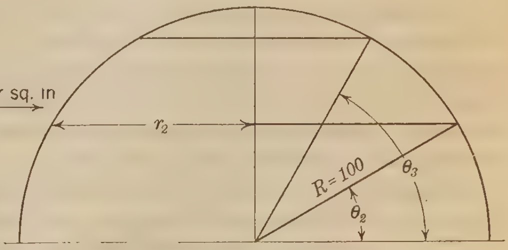

FIG. 9.

Shear Distortion of the Neutral Axis from the Base Up to the Level of θ_2.—There is a stress of 400 lb. in 8 in. by 12 in., or 4 lb. per sq. in. The section is about 400 in. high along the profile. The distortion equals $\frac{4 \times 400}{1\,000\,000} = 0.0015$ in.

Compression Distortion of the Horizontal Ring at the Level, θ_2.—The right half is compressed under an average stress of about 600 lb. per ft. of height, or 6 lb. per sq. in. of concrete; its length is 2 640 in. The total strain on the right half is $\frac{6 \times 2\,640}{2\,000\,000} = 0.008$ in. Similarly, the left side is strained, $\frac{2.5 \times 2\,640}{2\,000\,000} = 0.003$ in.

Therefore, the right half circle is about 0.005 in. smaller in arc length than the left, and at the same time has been moved toward the left about 0.00015 in. It is evident that these distortions are not alarming.

The distortions due to balanced loads and temperature may be treated here by a similar approximate method. If the dome and the ring at the base change their temperature equally, the distortion will obviously be slight. A temperature drop of 100° Fahr. on the dome only, would shorten the half profile $\frac{1}{1\,500}$, or about 0.85 in., relative to the ring. The average meridional dead load thrust is about 105 lb. per sq. in. Using a modulus of elasticity of 2 000 000, this gives a shortening of the half profile of about 0.07 in. The ring, if designed for 12 000 lb., will lengthen 1 part in 2 500, and r will increase in the same ratio, or about 0.48 in. The center will drop down, due to all these factors combined, a total of $(0.85 + 0.07 + 0.48)\,\frac{100}{27} = 5$ in.

A combination of the formulas for deflection and bending gives the formula,

$$\text{Stress} = \frac{4.8\,D\,E\,d}{l^2}$$

$$\frac{4.8 \times 5 \times 2\,000\,000 \times 8}{5\,760\,000} = 64 \text{ lb. per sq. in.}$$

At the same time the stretch of the ring, amounting to $\frac{1}{2\,500}$ will impose on the adjacent shell a tension of about $\frac{2\,000\,000}{2\,500}$, or 800 lb. per sq. in.

If there are horizontal bars in the shell, this action will cause them to assume part of the load of the base ring. In a stone dome it will open the vertical joints to a point where the dome will settle enough to keep the joints closed. Between this point and the ring the shell will have a conical surface of stress, tangent to the theoretical surface, and running to about ½ in. outside the theoretical point of support.

Example 2.—Consider a hemi-spherical dome of 100 ft. radius, built of 8-in. concrete weighing 100 lb. per sq. ft., and with a wind load of 10 lb. per sq. ft. of superficial area. (See Fig. 9.) The formulas used are Equations (61), (62), and (65), and the results are arranged for convenience in Table 3.

Distortion.—Shear Deflection of the Neutral Axis Up to the Level of θ_2.—The average shear $\left(\frac{1\,500}{8 \times 12}\right) = 15$ lb. per sq. in. The length of the arc is 660 in. The deflection equals $\frac{15 \times 660}{1\,000\,000} = 0.01$ in.

Tension Distortion in Right Half of Ring at Level of θ_2.—The average tension is 480 lb. on a section 8 by 12 in., or 5 lb. per sq. in. The length of the half ring is 3 250 in. The strain $= \frac{5 \times 3\ 250}{2\ 000\ 000} = 0.008$ in. expansion.

Similarly, the left side has about 0.008 in. of contraction. The right semicircle is then about 0.016 in. longer than the left one, and, at the same time, has been moved toward the right about 0.01 in. by shear deflection.

TABLE 3.—STRESSES DUE TO WIND LOAD.

y.	B_W.	A_W.	V_W.
$\tan\theta_1 = 0$; $\sec\theta_1 = 1$; $r = 100$ ft.; $e = 50$ ft.; $R = 100$ ft.; area = 63 000 sq. ft.; and $F = 630\ 000$ lb.			
+100	2 000	− 1 000	0
0	0	0	− 2 000
−100	− 2 000	+ 1 000	0
Dead load stress	+10 000	−10 000	0
$\tan\theta_2 = 0.57$; $\sec\theta_2 = 1.15$; $R = 100$ ft.; $r = 87$ ft.; $e = 25$ ft.; area = 31 400 sq. ft.; and $F = 314\ 000$ lb.			
+87	970	− 380	0
0	0	0	− 970
−87	− 970	+ 380	0
Dead load stress	+1 700	−6 700	0
$\tan\theta_3 = 1.74.$; $\sec\theta_3 = 2$; $R = 100$ ft.; $r = 50$ ft.; $e = 7$ ft.; area = 8 800 sq. ft.; and $F = 88\ 000$ lb.			
50	410	− 160	0
0	0	0	− 430
−50	− 410	+ 160	0
Dead load stress	−3 000	−5 600	0

RELATION TO THE ELASTIC THEORY

The exact theory of domes, as distinguished from the common theory as given in this and the preceding paper, involves the elastic deformations caused by the stresses, and their effect on the stresses themselves. A treatment of the elastic theory as applied to conoids with balanced loads may be found in a paper by Dr. Peter Pasternak,* of Zurich, Switzerland.

It is obvious that the elastic theory is of great value as a means of understanding the nature of the secondary stresses as well as for finding their exact amount. The common theory can be used with assurance only in those cases where the distortions are too small to be dangerous. The examples indicate a method of judging whether the dome can be designed safely by the common theory, or whether it must be studied by the more elaborate elastic formulas.

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* "Die practische Berechnung biegefester Kugelschalen, kreisrunder Fundamentplatten auf elastischer Bettung und kreiszylindrischer Wandungen in gegenseitiger monolither Verbindung," *Zeitschrift für angewandte Mathematik und Mechanik,* Band 6, 1926.

SUMMARY

In this paper the theory covering symmetrical loads and stresses for domes is extended to include formulas for vertical loads occupying one side of the dome only, the area above a given level, or the area below a given level. There is also a set of formulas for wind stresses.

These give stresses which are in most cases necessarily less than those for symmetrical loads, or else they may constitute so small an addition as to be negligible in practice. Their value is chiefly in providing a means by which the engineer who is faced with a dome of unusual size or shape may assure himself in regard to the order of magnitude of the neglected stresses.

These formulas, taken in this way as safeguards rather than as "exact" equations, constitute a complete practical theory for domes of all kinds having a homogeneous surface and a polygonal or circular plan.

DISCUSSION

A. FLORIS,* ESQ. (by letter).—It is rather doubtful whether a general theory of unsymmetrically loaded domes is much needed in a practical design of these structures. In domes the main loads are those due to their own weight and it is only in flat structures that the stresses produced by snow can be appreciable. Furthermore, domes which are relatively high in comparison with their span must be investigated also with regard to stresses caused by wind pressure. Any other loading condition is almost impossible in practice because domes, being mostly monumental structures, are symmetrical and symmetrically loaded, thus excluding in advance other possibilities. They possess not only symmetry of shape but, with the exception perhaps of wind pressure, symmetry of loading as well.

It is very fortunate that this is the case since the analysis of domes carrying eccentric loads arbitrarily chosen is not feasible to date. The theory of shells of rotation and especially of domes under axially symmetrical loading or wind pressure has received considerable attention abroad in recent years, and has been brought to a high degree of perfection. For reference, a bibliography of this subject is appended hereto. These investigations have revealed the important fact that the stress distribution in domes depends not only on their shape, but on the external loads as well, so that for any given load there is a stress variation corresponding to it.

These investigations have shown that the same law of stress distribution does not hold for various loading conditions. Navier's theory of bending as developed for prismatic bars cannot be assumed to apply in the case of domes. For this reason the author's fundamental assumption that the law of the trapezoid will be valid in the case of unsymmetrically loaded domes, which are systems in space, is more than a rough approximation; it is absolutely impossible to apply. Consequently, formulas derived under this assumption are of doubtful value in the design of domes and similar structures.

In addition, the author treats the analysis of wind stresses in a dome under the same assumption of the planar theory of flexure. The problem of wind stresses in domes has been solved by other writers for a certain distribution of wind pressure only. Assuming a pressure on the windward, and a suction on the leeward, sides of the dome, the stresses thus produced can be analyzed if the external loads are expressible in a Fourier series. They can be determined either by solving the differential equations of the problem, or, more simply, by statics, as has been shown recently by F. Dischinger.† This special wind loading with pressure on one and equal suction on the other diametrically opposed sides of the dome is sometimes called "anti-symmetrical".

Theory of Stress Distribution in Beams with Anti-Symmetrical Wind Pressure.—The thickness of a dome is usually small as compared with the dimensions of rise and span, so that it can be considered as a shell generated about an axis of rotation. In such a shell the stresses due to the external loads are, under certain conditions, membrane stresses, that is, they are axial, acting in its mid-surface so that they can be determined by statics. This stress analysis for axially symmetrical loads was first given by Lamé and Clapeyron* more than one hundred years ago and since then it has been repeatedly used by many writers. The stresses due to the boundaries (support, change of curvature, etc.) of the shell are caused mainly by bending moments and must be determined by taking into account the deformation of the structure. They are statically indeterminate and, at the same time, are independent of the external loads.

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* Civ. Engr., Los Angeles, Calif.

† "Schalen und Rippenkuppeln," von Franz Dischinger, Handbuch für Eisenbetonbau, Vol. 6, Pt. 2, Fourth Edition, Berlin, 1928, p. 197.

In a shell of rotation, with an axially symmetrical loading, the meridian and ring stresses are membrane stresses and, since they are normal to the corresponding sections of the shell, they are principal stresses. In an anti-symmetrically loaded shell of rotation, on the other hand, the meridian and ring stresses, although they are membrane stresses, do not act normal to the sections of the shell, but are inclined. By taking the components of these stresses, shearing stresses acting in the mid-surface of the shell are produced, in addition to the normal (meridian and ring) stresses. In the case of an anti-symmetrical loading the stresses due to the boundaries of the shell differ from those caused by an axially symmetrical loading.† However, they can be determined approximately by using the methods applied to such loading.‡

Neglecting the stresses due to the boundaries of the shell the rigorous analysis of the membrane stresses in a spherical shell has been given by H. Reissner§ in the case of loads expressible in a Fourier series. He has determined these stresses for a wind pressure following the sine law. Later, Dischinger|| discovered that, although this pressure distribution which is commonly assumed in design, is greater than the square sine law, nevertheless, the stresses produced by the former were found to be smaller than those due to the latter. This is true, however, only in the spherical shells. In shells with a polygonal base the greatest wind pressure produces also the greatest stresses, especially meridian stresses. These shells with polygonal rings but with curved meridians are a new type of dome possessing very favorable and, at the same time, interesting properties.

The important result in Dischinger's investigations as applied to the paper under discussion is the fact that only for the sine law of the wind pressure are the stresses in a spherical shell linearly distributed¶ In this case alone is there a neutral axis of the meridian—and ring—stresses, which is perpendicular to the wind direction. (See Fig. 10 (*a*).) The intensity of the

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* In "Mémoires présentées par divers savants," No. 4, 1928, p. 465.

† "Ueber Spannungen in symmetrisch und unsymmetrisch belasteten Kugelschalen (Kuppeln), insbesondere bei Belastung durch Winddruck," von E. Schwerin, Thesis, Berlin, 1918; also, abbreviated in *Armierter Beton*, No. 12, 1919, p. 25.

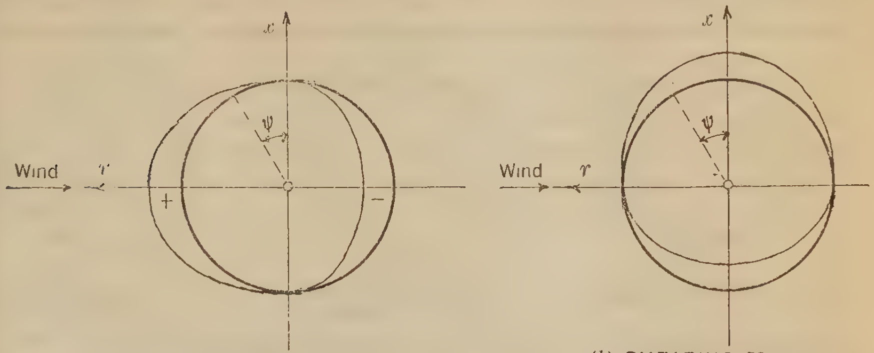
‡ "Schalen und Rippenkuppeln," p. 247.

§ "Spannungen in Kugelschalen (Kuppeln)," von H. Reissner, in Müller-Breslau Festschrift, Leipzig, 1912.

|| "Schalen und Rippenkuppeln," p. 200.

¶ *Loc. cit.*, p. 198.

shearing stresses, on the other hand, reaches its maximum in the neutral axis disappearing in the points where the normal stresses have their greatest values (see Fig. 10 (*b*).) In all other cases of anti-symmetrical wind pressure that follow various sine laws, the spherical shell possesses several neutral axes which not only do pass through the center of the parallel circles, but also connect the points of intersection of these axes with the circumference. (See Fig. 11 (*a*).) In this case, therefore, the distribution of the meridian and ring stresses is no longer linear. (See Fig. 11 (*b*).) The shearing stresses have their maximum values in the zero points of the normal (meridian and ring) stresses and *vice versa*. (See Fig. 11 (*c*).) Thus, for the various laws of wind-pressure distribution there are a certain number of corresponding neutral axes or points of change in the signs of stresses. For a few cases, these are as follows:



(*a*) MERIDIAN AND RING STRESSES (*b*) SHEARING STRESSES

FIG. 10.—STRESS DISTRIBUTION IN A HORIZONTAL CROSS-SECTION OF A SPHERICAL DOME FOR A WIND PRESSURE FOLLOWING THE SINE LAW.

Wind pressure variation.	Neutral axes.
$\sin \psi$	$1 = 1$
$\sin 3\psi$	$3^2 = 9$
$\sin 5\psi$	$5^2 = 25$
……	……
$\sin n\psi$	n^2

In this list $\sin \psi$ corresponds to the sine law of wind pressure, while $\sin 3\psi$ expresses the square sine law of this pressure*.

The author apparently does not realize the great importance of the pyramoidal domes, as he calls those with a curved meridian and a polygonal base. Thus, he treats an entirely new type of dome unaware of its remarkable properties with their far-reaching consequences in structural engineering. In these polygonal shells, as Dischinger was enabled to show, the stresses in any given point can be found from those developed in an inscribed spherical shell.† The "hips", that is, the intersection of two adjacent sectors of these shells, remain free from bending stresses for axially symmetrical as well as anti-symmetrical loadings, provided the structure possesses a symmetry of shape. While in the shells of rotation the loads are transmitted to the abutment by the aid of the meridian stresses, the loads in the polygonal shells are sustained partly by the shell itself and partly by its "hips". In the polygonal shells, therefore, there are developed purely membrane stresses, a fact which is very important from the point of view of economy. These shells which are stressed mainly axially can have very thin sections because their weight does not increase appreciably with increasing span, as is the case, for instance, in rigid frames and similar structures. This discovery combined with that of the self-supporting arch barrel* is the latest successful application of the advanced theory of statics to the problems of structural engineering.

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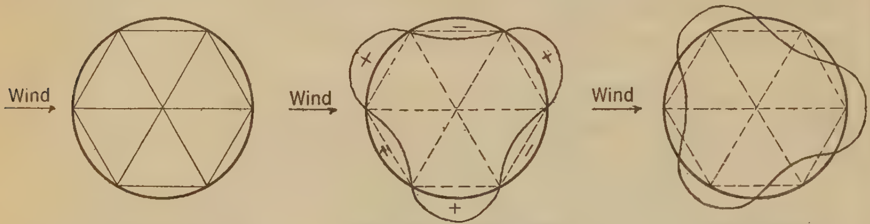
* "Schalen und Rippenkuppeln," pp. 200 and 208.

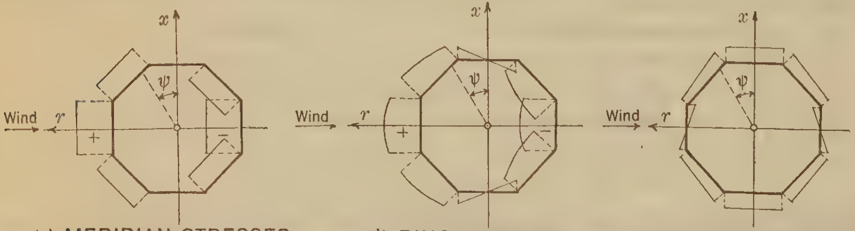
† "Die Theorie der Vieleckkuppeln und die Zusammenhänge mit den einbeschriebenen Rotationsschalen," von Franz Dischinger, *Beton und Eisen*, 1929, No. 5, pp. 100-107; No. 6, pp. 119-122; No. 8, pp. 150-156; and No. 9, pp. 169-175.



(*a*) NEUTRAL AXES (*b*) MERIDIAN AND RING STRESSES (*c*) SHEARING STRESSES

FIG. 11.—STRESS DISTRIBUTION IN A HORIZONTAL CROSS-SECTION OF A SPHERICAL DOME FOR A WIND PRESSURE FOLLOWING THE SQUARE SINE LAW.

With regard to the stress distribution of the polygonal shells the results of the theory developed by Dischinger for the anti-symmetrical wind pressure following the sine law will be sketched briefly. Owing to this law of pressure variation these shells have but one neutral axis.



(*a*) MERIDIAN STRESSES (*b*) RING STRESSES (*c*) SHEARING STRESSES

FIG. 12.—STRESS DISTRIBUTION IN A HORIZONTAL CROSS-SECTION OF A PYRAMOIDAL DOME FOR A WIND PRESSURE FOLLOWING THE SINE LAW.

The meridian stresses within a sector between two hips are constant and vary only from sector to sector according to the sin ψ law (see Fig. 12 (*a*)). Consequently, by assuming that the resultants of the meridian stresses are acting in the middle of these sectors, the same law of stress variation is valid as in the shell of rotation.

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* "Schalen und Rippenkuppeln," p. 269; "Die Schalendächer des Elektrizitätswerkes in Frankfurt a. M.," von U. Finsterwalder, *Beton und Eisen*, 1928, pp. 205-209; "Die Schalenbauweise Zeiss-Dywidag," von Fr. Dischinger und U. Finsterwalder, *Bauingenieur*, 1928, Nos. 44 to 46; also, "Self-Supporting Arch-Barrrel," by A. Floris, *Western Construction News*, April 10, 1929.

Two kinds of shearing stresses are produced, namely, those due to the beam action of the shell between two adjacent hips and those caused by the dome action of the shell itself. The maximum values in the first case are reached in the sector, $\psi = 90^\circ$, while those of the second case take place at $\psi = 0$. The combined diagram of these two kinds of stresses is represented in Fig. 12 (*b*).

The ring stresses in a sector due to the beam action are composed of a parabolically and a linearly distributed stress. Both these stresses are zero at $\psi = 0$ and maximum at $\psi = 90$ degrees. The linearly distributed stress is constant within a sector. On the other hand the ring stresses due to the dome action of the shell, which are similar to those of a spherical shell, vary from hip to hip according to the sin ψ law. However, the difference in this case will be that the ring stresses remain constant within a sector and have as in spherical shells a maximum value at $\psi = 90^\circ$, while they vanish at $\psi = 0$. The addition of the ring stresses is shown in Fig. 12 (*c*). With an increasing number of sides in a polygonal shell the ring stresses approach, but remain always greater than, those of a shell of rotation.

The properties of the polygonal domes herein outlined are manifestly of outstanding importance for the future development of reinforced concrete structures. Due to the combination of beam and dome actions in these shells the loads can be transmitted to the foundations by the aid of columns only, without the aid of intermediate supporting members, walls, beams, or arches, to a maximum distance of 130 ft. approximately between columns.

It is very doubtful therefore, whether domes designed with ribs subjected to bending can compare favorably with the shells of rotation or with those of a polygonal base which have to sustain largely axial stresses. The remarkable self-supporting ability of the polygonal shells is best illustrated by the fact that each of the three reinforced concrete domes of the Great Market in Leipzig, Germany, has an inscribed diameter of 245 ft., a maximum thickness at the abutment of 4¼ in., and weighs only 2 380 tone.[*]

In conclusion, the writer wishes to express his regret for having criticized this practical and carefully prepared analysis of the unsymmetrically loaded domes. In doing so he intended merely to state known facts which have possibly escaped the attention of the author.

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[*] “Die Grossmarkthalle in Leipzig, ein neues Kuppelbausystem Zusammengesetzt aus Zeiss-Dywidag-Schalengewölbe,” von Franz Dischinger und Hubert Rusch, *Beton und Eisen*, 1929, No. 18, pp. 326-329; No. 19, pp. 341-346; and No. 23, pp. 422-429.

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DAVID C. COYLE,* M. AM. SOC. C. E. (by letter).—The tendency of American engineers to insist on rough and "practical" formulas for use in actual work is naturally irritating to many who are accustomed to the everyday employment of the elastic theories, for example, those who have been trained in Continental Europe. There is, however, some little justification for roughly approximate methods, a fact which was brought out in the discussion of the paper by Albert W. Ross, Jr., Jun. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., in 1928, on the modified slope-deflection theory in wind bracing.*

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* Cons. Engr., New York, N. Y.

Briefly, the writer's point of view is one of frank though respectful skepticism as to the relation between the admirable mathematical systems propounded and the data to which they are applied. Concrete on the job is so likely to have occasional lapses from that perfect homogeneity which the elastic theories presuppose. As for the n neutral axes which would exist in a spherical dome anti-symmetrically loaded with a wind of the degree, $\sin n \psi$, there is no probable reason to doubt the mathematics of the matter, but it is questionable whether in actual construction a wind of such a special character needs to be considered.

The difference in point of view is very well illustrated by Mr. Floris' comment on the trapezoidal distribution of pressures due to unbalanced loads, which he quite correctly states has no logical basis in its analogy to the stresses in a prism. The fact remains, however, that the stresses obviously have a distribution which is of the same general form as the assumed distribution; a reference to his Figs. 10 and 12 will indicate at once that the results obtained by advanced statics are indistinguishable at small scale from those shown by the rough formulas. It is evident that the difference between the trapezoidal distribution and any other probable distribution of the stresses is similar to that between a five-centered oval and an ellipse. The mathematics of the two are very unlike, but the numerical results come very close to each other. As for the wind effects shown in Fig. 11, it is permissible to wonder whether the purity of the mathematics may not have obscured the inconsequence of the practical application.

The older theories of domes on the Continent seem to have involved originally the assumption of bending moments in the ribs of a pyramoid. No doubt the development of advanced statics which has justified mathematically the use of a center-line stress system has seemed to be a radical liberation. It may be fair to remark, however, that the common theory in the United States has always been too inexact to take the secondary bending into account. For this reason Mr. E. Schmitt's formulas for pyramoids, published in 1904† were based on the absence of bending in the ribs. The further elimination of bending from the shell between the ribs or "hips" is interesting, but the results are perhaps unfortunate. There is a constant danger that the growth of mathematical methods, the accuracy of which exceeds the homogeneity of the materials, may lead to economy which is based on an illusion. The domes of the Great Market in Leipzig, Germany, are illustrative of this point. By a mathematical process the builders have convinced themselves that practically no thickness at all was required. Similarly, it has always been recognized that the theoretical required thickness of a spherical dome is practically zero, the live load being small. It has not followed, however, that zero thickness was the best for the purpose.

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* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1395.

† *Transactions*, Am. Soc. C. E., Vol. LII (1904), p. 293.

In the building of actual structures, the proper function of mathematics is to check the general validity of the practical formulas used for the work itself; but if the accuracy of the mathematical methods used is allowed to become the main object, it is dangerous. The stresses having been computed to four places of decimals, what if a storm blows the covers off some night and the previous day's pour is frozen? Or what if there is sugar in one load of sand? Or what if two out of sixteen footings settle $\frac{1}{8}$ in. more than the other fourteen? Or how about the indications that in some concrete the ratio between concrete and steel stresses is not equal to n, but to a function of time? What happens to the elastic theory if n is incorrect?

It is evident that the formulas given in this paper have a real approximate relation to the actual stresses in the structure. The close correspondence of the general shape of the stress diagrams with those of the more accurate theories indicates that the paper is not widely at variance with the facts. If their algebraic derivation is free from error, these formulas are suitable to give the engineer an idea of whether his design is well on the safe side. At the same time, they are free from the disadvantages of the high mathematics, of taking an unconscionable time, and of producing the illusion that their results are more accurate than their data.

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Paper No. 1754

ENGINEERING APPLIED TO NATIONAL PARKS*

By Stephen T. Mather,† Esq.

With Discussion by Messrs. H. K. Bishop and W. W. Crosby.

Beginning in 1872, with the setting aside of the Yellowstone National Park "as a public park or pleasuring ground for the benefit of the people", the fundamental principle underlying the National Park System has been the preservation of scenery—the natural objects and wonders and their retention in their natural condition. Congress re-affirmed this principle in creating the National Park Service in 1916, by stating‡ that the Service, in its administration of the National parks and monuments, was "to conserve the scenery and the national and historic objects, and the wild life therein, and to provide for the enjoyment of the same in such manner and by such means as will leave them unimpaired for the enjoyment of future generations".

In order that the people might enjoy the National parks it was necessary that roads be constructed to and within them to permit access to their treasure troves. At first, these roads were haphazard wagon trails, in existence at the time of the reservation of the parks, and it is the conversion of these old wagon roads into adequate automobile highways, or the construction of new roads, that forms one of the principal engineering problems confronting the National Park Service. This does not mean the establishment of a gridiron of roads, nor over-development. Large areas must always be preserved in their wilderness character.

Development of the Yellowstone Road System

Due to the foresight and engineering ability of the late Gen. H. M. Chittenden, U. S. A. (*Retired*), M. Am. Soc. C. E., who, as an officer in the Engineer Corps, was placed in charge of the road work in Yellowstone Park in 1883, the system of roads laid out in that Park is now a standard for general rational park road development. The main loop road, which embraces in a general circuit all the important centers of interest in the Park, and the connecting gateway roads, are the work of General Chittenden. This system provides 303 miles of roads in an area of 3 348 sq. miles. For a number of years after 1883, Congress annually appropriated $40 000 for road work, part of which was spent in actual construction and the remainder in the preparation of plans and in the making of surveys. Finally, in 1902, Congress adopted a definite program of road work and appropriated the sum of $250 000 per year for three years.

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* Presented at the Joint Meeting of the City Planning and Highway Divisions, New York, N. Y., January 19, 1928.

† Director, National Park Service, U. S. Dept. of the Interior, Washington, D. C. Mr. Mather died on January 22, 1930.

‡ Act of August 25, 1916 (39 Stat., 535) establishing the National Park Service.

While the road system laid out by General Chittenden is still in use (1928), it was not constructed to bear the modern vehicular traffic it now sustains. That it could continue to be used by automobiles after the opening of the Park to motor traffic in 1915 speaks volumes of praise for his engineering ability, and indeed as an engineer he was ahead of his day and age. Many of the roads laid out by him will continue to be used after they have been widened and surfaced. Only in places will changes have to be made in grade and alignment to bring them up to modern standards. It is doubtful whether extensions of the present road system to regions not now served by roads will be needed.

Extension Throughout National Parks

Federal road construction under the Army Engineer Corps was extended to Crater Lake and Mount Rainier National Parks, but then came to a standstill. When the writer first began, in 1915, to co-ordinate National Park work, he was met with decided views on park road building in the Appropriations Committee of Congress, the members of which did not realize the importance of adequate highways. One Chairman of a House Appropriations Committee suggested that 14-ft. dirt roads, with frequent turnouts for passing vehicles, would serve park needs amply. It was not until members of the Committee had ridden over the old narrow road from El Portal, Calif., into Yosemite Valley, that the first appropriations for rebuilding this road were forthcoming.

During the fiscal years 1917, 1918, and 1919, a total of $160 000 was received for the El Portal Road. From this time until 1924 only small annual appropriations were available for road construction, scattered in the separate park appropriations, while outside the parks, the States with Federal Aid and the Federal Government in the National Forests were building modern approach roads. These roads were bringing increasing numbers of motorists to park boundaries, only to find poor roads to bear the concentrated traffic from the far superior approach roads.

Beginning of Park Road Program

Under these conditions the road situation was becoming very acute. Therefore, in 1924, the Park Service presented to Congress a road program calling for authorization of appropriations of $2 500 000 per year over a three-year period. This bill passed the House and Senate unanimously and became a law. In the first year, however, the National Park Service was permitted to submit a deficiency appropriation of only $1 000 000, and even this sum was temporarily lost because of the defeat of the deficiency measure by a filibuster in the closing hours of the last session of that Congress. In December, 1924, the new Congress passed the deficiency bill with the item of $1 000 000 for park road work and in the spring of 1925 the first road contracts were let.

The plans had been prepared in 1922, but actual work did not commence until 1925. Meanwhile, road standards had changed rapidly. States which previously had led the Nation in road building were beginning to rebuild their highways to accommodate the motor traffic that had been created with the popularization of the automobile. It is perhaps just as well that funds were delayed, because the Park Authorities were able to profit by the mistakes of others. Higher standards were adopted for park roads and previous estimates were scrapped.

After a short working period, it became clear that, for the Park Service to handle its road building properly, it would need an engineering force of high caliber. The most competent highway engineers had been developed by the United States Bureau of Public Roads and were either in its employ or with State highway organizations. The U. S. Bureau of Public Roads was then busy in its Federal Aid and other work, getting into mountain road construction comparable to conditions found in the National parks. In the interests of Federal co-operation and co-ordination and the elimination of duplication and waste, an agreement was made between the Secretaries of the Interior and Agriculture whereby the resources of that splendid engineering organization, the Bureau of Public Roads, were made available for the carrying out of the park road program.

It is a splendid working agreement. Since 1925, the Bureau and the Park Service have spent or obligated a total of $9 000 000. They have "taken stock" of the problem and know what they have ahead. Preliminary plans based on reconnaissances have been prepared and presented to the Bureau of the Budget, and to the Appropriations Committee of Congress. The program calls for an ultimate expenditure of $50 000 000. The House of Representatives, in passing the Interior Department Appropriation Bill on January 13, 1928, has made $5 000 000 available for the year 1928-29, and if the appropriations are continued on this basis, the road program will be completed in ten years.

This program does not involve gridironing the parks with roads. It contemplates the reconstruction and surfacing of nearly all old existing roads and the construction of only a few new ones. Nearly all the parks are located in high mountainous regions where road construction is an expensive proposition and where the scenery must be conserved and at all costs left as little scarred as possible. That is the job of the Bureau and the Park Service and that is where the trained Landscape Force of the Park Service is indispensable.

Landscaping of Park Roads

One important requirement is the preservation of park landscapes. In the lower elevations, rigid standards calling for railroad cuts must give way to rolling grades; straight alignments must be sacrificed for graceful curves where trees or other beauty spots should be saved (Fig. 1). There is no necessity for speedways in the parks. Bridges must fit into the landscape. Where construction must be carried along sheer mountain sides (as in Fig. 2) the excavated material cannot be dumped over to scar the whole terrain. Contracts must provide for end-hauling where necessary, and contractors and sub-contractors must be watched to see that they comply with this provision. Rock is being blasted from a cliff along the old Garden Wall Trail at Glacier National Park as shown in Fig. 3. This is typical of some of the difficulties encountered.

The proposed Big Oak Flat Road reconstruction in Yosemite, a contract for the park section of which will be let early in 1929, will be specially guarded in this respect. This road ascends the north wall of the Yosemite Valley. Permitting the excavated material to be dumped off the right of way and to crash down the precipitous granite walls would cause a scar that Nature would require generations to efface. End-hauling on this project will increase the cost, but spread out over a hundred years this will be very little, and compared to the value of the magnificent scenery the cost is infinitesimal.

An example of what it is possible to accomplish along this line can be noted in the work completed in Yellowstone Park in 1927. Years ago, when funds were limited, Army engineers started to build a cut-off between Madison Junction and Old Faithful down the Firehole River. This project is located through a precipitous canyon which affords wonderful views of the rushing river. The work was partly completed, but the excavated rock was tumbled down into the river, making an unnatural ragged bank. Ten years later, when the final mile was completed by park force account, authority was given to preserve the natural beauty of the canyon. A dry rock wall was built up from the river; cut and fill were balanced. The contrast between the old and the new work shows that the cost of $100 000 for this mile of construction was more than justified. On some of the famed roads of Switzerland, it is this attention to landscaping detail—to making the road fit into the picture—that has helped year after year to bring visitors from all parts of the world to enjoy the beauty of the Alps.

Trees must be protected, and side-hill cuts must be made with a consideration as to whether they will become covered with fern, or flower, or shrub. Parapets and rock walls must be designed to harmonize and yet give a feeling of safety in high places.

THE ZION PARK ROAD

The Zion-Mt. Carmel Road is one of the most important in the park program. It links the scenic attractions of Southwestern Utah, including Zion National Park, Bryce Canyon, and the North Rim of the Grand Canyon National Park. It reduces the distance between Zion and Bryce from 165 miles to 85 miles and between Zion and the North Rim from 150 miles to 130 miles. It ascends from the floor of Zion Canyon to the East Rim—a rise in elevation of nearly 4 000 ft.—in a distance of 8.75 miles. Contracts for Sections 1

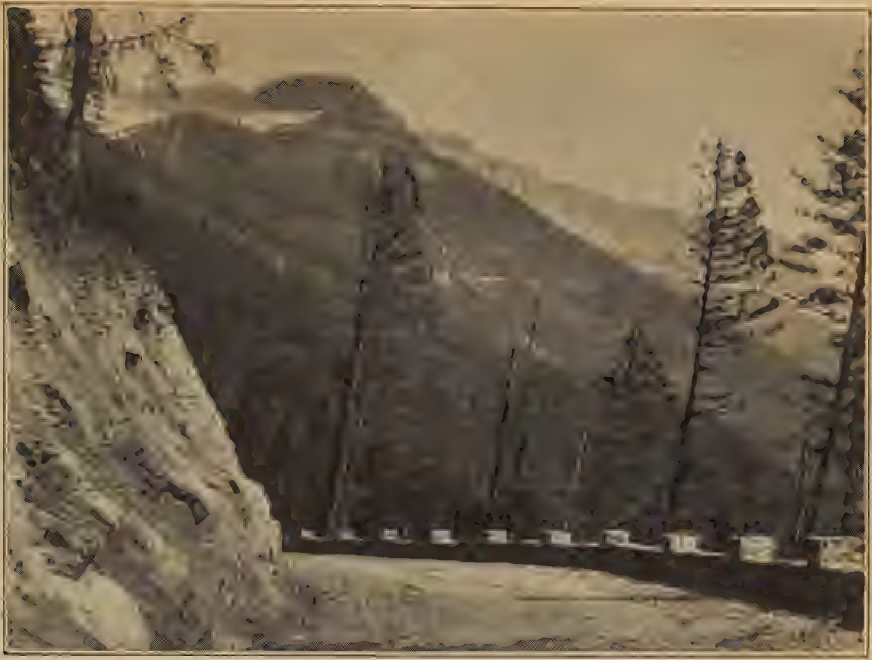

FIG. 1.—CURVED ROAD WITH MASONRY GUARD-RAIL, GLACIER NATIONAL PARK.

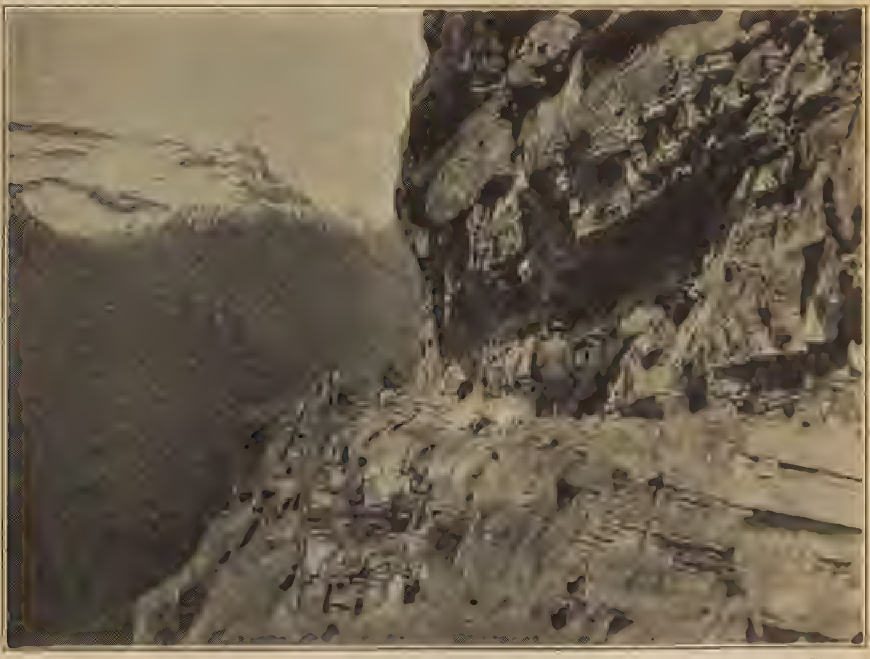

FIG. 2.—CONSTRUCTION WORK ON THE TRANS-MOUNTAIN HIGHWAY, GLACIER NATIONAL PARK.

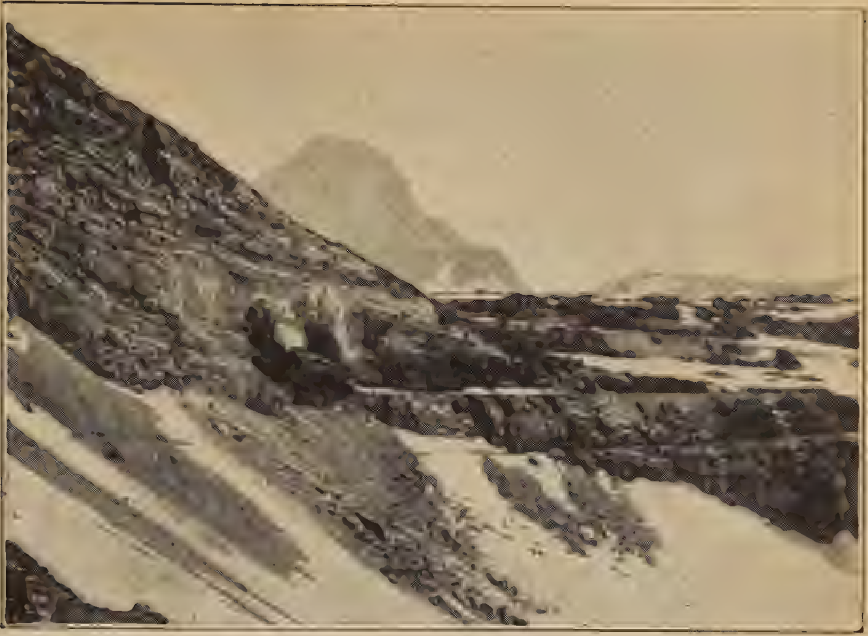

FIG. 3.—ROCK BLASTING IN GLACIER NATIONAL PARK.

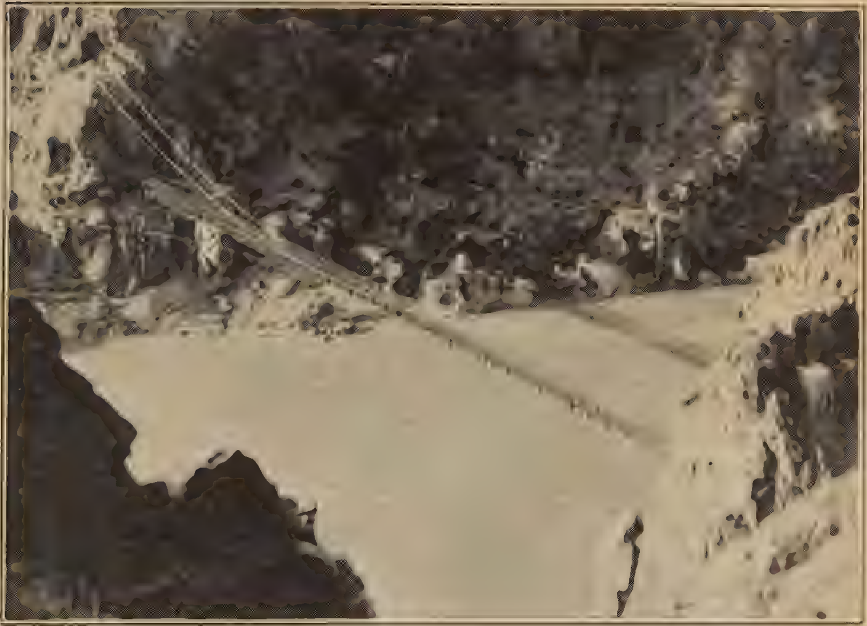

FIG. 4.—KAIBAB TRAIL BRIDGE.

and 2 of this project were let in September, 1927, calling for the construction of 5.02 miles, including 5 582 lin. ft. of tunnel excavation. The cost of this section with an additional small contract will be $800 000. It is estimated that the remainder of the project (3.65 miles) will cost $450 000, or for the total construction, $1 250 000. In addition, the State of Utah is constructing 16 miles of road connecting the east boundary of Zion Park with the Town of Mt. Carmel.

Another interesting feature of park road work is found in Mesa Verde National Park, Colorado, where Navajo Indians constitute the bulk of the laboring forces. There is never any difficulty in securing sufficient Navajo help, for when they get word that a construction job is to be started, they drift into the park, build their little "hogans" up against a near-by cliff, and are ready to start work when needed. Some of these same laborers, after the day's work is done, give sections of their "Yeibachai", or medicine dance, at the evening camp fire.

Parking of Automobiles

With the construction of roads comes the ever-increasing problem that all communities to-day are trying to solve—the parking of automobiles. In a restricted area like Yosemite Valley, this has become one of the most important of immediate problems. Mr. Frederick Law Olmsted made a preliminary study of parking conditions in Yosemite Valley. As a result he suggests*:

> "* * * a principle which ought to be applied generally in the Valley, namely: That wherever there is occasion for the concentrated parking of a considerable number of cars, space should, as far as practicable, be provided for them, having these conditions: (1) Entirely separate from, and screened from, the roads used in moving about the valley; (2) so arranged that no parked car will be blocked in by subsequent arrivals; (3) so interspersed with trees (pre-existing or planted for the purpose), protected from injury (by the use of posts or racks or other guards close to the trunks), that the space and the cars will be largely covered by the foliage and unnoticeable in the plunging views into the valley; and (4) when the limit of such parking is reached, no further parking to be permitted in the vicinity."

Snow Removal

Another pressing problem is connected with the removal of snow from the high mountain passes. Since opening dates for the various parks are announced several months ahead, and railroad tickets to the parks are sold on this basis, it is necessary that roads be cleared of snow sufficiently to permit traffic over the high mountain passes before the opening date. In Rocky Mountain National Park, snow removal is a difficult task, because the Fall River Pass, with an elevation of 11 797 ft. (the highest in the National Park System), must be opened to travel on June 15 of each year, which is the official opening date. In 1927, the 7 miles of road east of the Pass were opened by a steam shovel with a special snow dipper, while 6 miles beyond the west side of the Pass were opened by hand shoveling.

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* From unpublished informal memorandum submitted by Mr. Olmsted to National Park Service Official.

Sylvan Pass, in the Yellowstone, also presents a difficult snow-removal problem. In 1927, due to the heaviest snowfall in years, more than 15 tons of explosives were used in clearing snow from Sylvan and Dunraven Passes, the Mount Washburn Loop, and the South Entrance Road. Crater Lake also had a serious problem, and the last impassible snow on the rim road was not removed until the middle of August, 1927. Sections of the river road must be rebuilt to avoid these heavy drifts of snow.

DUST PREVENTION

Until the parks roads are rebuilt, there will be the problem of alleviating the intolerable dust nuisance on some of the existing roads. An innovation in maintenance work in Yellowstone Park in 1927, was the substitution of a light road oil, instead of water, to lay the dust. This was a much more efficient method, although more costly. The entrance roads and the main loop were oiled for a distance of 126 miles with from $\frac{1}{9}$ to $\frac{1}{2}$ gal. per sq. yd. Old wooden tank trucks were used to haul the oil from cars to the distributor and about twenty wooden storage tanks were set up and used to store the oil. The water sprinkling system is being dismantled as rapidly as possible, as the tanks were an eyesore along the road. Four of the steel ammunition bodies that came with the trucks from World War surplus were covered with steel plates and welded and mounted on trucks and used to haul oil. Nine tank trucks and one distributor truck were used during June, July, and part of August, in transporting and spreading approximately 300 000 gal. of road oil.

SANITATION

One of the outstanding perplexities has been the providing of the necessary sanitary arrangements and safeguards for the traveling public. In the early days of Park Service history, when there were comparatively few visitors, common earth toilets and the burning of garbage in open pits—after first allowing the bears to work over it as an attraction to the tourist—were found to be sufficient. In the more modern days, however, this problem has the same phases as in large cities. Toilet facilities are being gradually changed to the most modern of city systems and garbage is being disposed of by especially designed incinerators. In conjunction with the garbage disposal, there are can-mashers to reduce the refuse cans to small bundles of unrecognizable metal, which are easily disposed of in such small compact masses.

The water systems are modern in every way. Originating in undefiled mountain streams and springs, the water is safeguarded by frequent analyses, and every precaution is taken to keep it free from contamination until it reaches the consumer. Inspections of the culinary arrangements in all hotels, camps, and concessions where public food is handled and sold are made regularly. In Grand Canyon National Park, where the water supply is a serious problem, and water has to be hauled for many miles in tank cars, a system has been perfected which purifies the waste water to the extent that it may be safely used over and over for flushing toilets, washing automobiles, and for many other non–domestic purposes. This effects a saving of thousands of dollars each year in haulage costs alone.

In Mesa Verde National Park where there is no live water during the summer the Park Service authorities have had to resort to the development of the water supply by catchment and storage. One unit has been constructed which provides for a corrugated, galvanized-iron catchment area, 1 acre in extent, with two steel frost-proof storage tanks, each with a capacity of 125 000 gal. The catchment area gathers, by means of a flume, the snow and rain falling thereon and conducts them through a rapid sand filter to the steel storage tanks. These steel tanks have not proved entirely satisfactory because when warmer weather prevails the water has a decided flatness and taint, possibly due to pollen from pinon trees. Funds have been included in the new Appropriation Bill for the construction of a second unit, in which concrete storage tanks will be substituted for the steel tanks. Plans for this work are now (1928) being drawn.

All this sanitary work has been accomplished by the able assistance and splendid co-operation of the expert sanitary engineers of the United States Public Health Service.

Power, Telephone, and Telegraph

In many of the National Parks there are hydro-electric systems which furnish light, power, and heat for administrative purposes within the boundaries. In numerous instances where it would be difficult, if not impossible, to obtain electricity from other sources, the surplus is furnished to the operators of the public utilities at reasonable rates. Most of the parks are abundantly supplied with possibilities for development along these lines, and local systems to supply the needs of a park are being augmented from time to time as funds become available. These problems are handled individually and frequently are difficult to solve since neither water-falls nor cascades can be used, and all installations must be made so as to mar the scenery as little as possible.

In order to provide adequate long-distance telephone service in Yellowstone, the American Telephone and Telegraph Company, using the Hotel Company and National Park Service pole lines, strung a two-wire, No. 128, copper, metallic circuit from Mammoth Hot Springs to Old Faithful and West Yellowstone, and another circuit from Mammoth Hot Springs to Yellowstone Lake *via* the Grand Canyon, and new magneto switchboards were placed at Mammoth Hot Springs, Old Faithful, the Grand Canyon, and Yellowstone Lake. These new copper circuits are built through to Helena, Mont., and give the park high-class, exclusive service. The new system when completed will cost about $150 000.

In the Carlsbad Cave National Monument, New Mexico, there is an entirely different electrical lighting problem. This is the largest and probably the most beautifully decorated cave known in the world. The trip through the cave requires about five hours and proper lighting is of paramount importance. Congress is granting funds for this work and members of the Appropriations Committee who have visited the Caverns have taken a great interest in the problem. They have suggested that Park Officials consult with some of the best known electrical and stage lighting experts, with a view to developing a system so that the lights will come on slowly until full illumination is obtained and then gradually die down to add to the impressiveness of the scene.

TRAIL CONSTRUCTION

Great wilderness areas must be retained in all the parks to be entered only by trail. These are for the horseback rider and hiker, as well as the administrative forces on game and fire patrol. Trail-building plans must go hand in hand with the road-building program. Some trail construction that is not equalled anywhere else in the world has already been completed. The trail across the Grand Canyon, according to travelers familiar with all parts of the world, is the best construction to be found. A view of Kaibab Trail Bridge is shown in Fig. 4. The lower cable is provided to support the suspension bridge against wind.

DIVERSE ENGINEERING PROBLEMS

This paper illustrates some of the diverse engineering problems which confront the National Park Service. They involve the expert services of the highway engineer, the landscape engineer, the sanitary engineer, the electrical engineer, and the mechanical engineer. A point has now been reached in park development where the same problems are encountered as in town planning.

To a great extent, visitors in the National Parks are gregarious and desire to herd together in the large public camp grounds. Streets must be laid out, sanitary conveniences located, water lines installed, and community buildings erected, where visitors may gather during inclement weather and in the evenings. Incidentally, these community buildings have become centers for educational work. With the increased crowds comes the necessity for hospitalization. In a few of the parks there are already some hospital facilities and these are now being extended into other parks.

CONCLUSION

The writer would also like to mention wild life preservation, and methods of combating insect infestations by means of power sprayers and in other ways, because although park work is primarily based on preserving the natural features of these reservations while at the same time developing them for tourist use, engineering along some lines is an important factor in nearly every phase of the Park Service activities. Park Officials could not continue to serve the public were it not for the engineers.

DISCUSSION

H. K. Bishop,* M. Am. Soc. C. E. (by letter).—The author has covered this subject so thoroughly that little has been left to discuss. The writer agrees entirely with Mr. Mather.

The U. S. Bureau of Public Roads has benefited greatly by its co-operation with the Park Service. In the beginning the personnel of the Bureau was perhaps a bit inclined to think too much of economy in road construction and too little of scenic effects and the preservation of the beauty of the country through which the roads ran. Through the National Park Service the Bureau has been brought into contact with landscape engineers, and consequently a new appreciation of scenic factors will be evident in the character of the roads built hereafter in the National Parks and National Forests of the West.

The writer has a great admiration for the work that the Park Service is doing. Those who have visited the Western parks will appreciate it. The Service is opening trails and roads to scenes of beauty that were hitherto inaccessible.

Mr. Mather mentioned two projects: The Transmountain Road in Glacier National Park, and the road in Zion National Park, each of which will cost more than $1 000 000. The State of Arizona is now building a bridge across the Grand Canyon at Lee's Ferry which will change entirely the situation in that part of the United States. The impassable canyon that heretofore has cut off highway travel between the north and south and, to some extent, between the east and west, will be bridged at Lee's Ferry, and a road will probably be constructed from Flagstaff, Ariz., to Zion National Park, Bryce Canyon, and Salt Lake City, Utah. The writer predicts that this road, in time, will be one of the heavy traffic roads of the West.

Mr. Mather describes the snow-removal problem. There has been a great deal of trouble in opening some of the passes through the Rocky Mountains in time for the traffic in the spring. Great strides have been made since 1925, but snow removal work is still a new problem in the parks.

In the Western States the best road surfacing is obtained from fine crushed rock or gravel; but experience has taught the Bureau of Public Roads that it must do something to eliminate the dust which becomes a serious nuisance when the traffic increases. For that purpose a method has been developed of mixing a road oil in place, which is believed to be something new in road building. After the crushed rock road has been built and traffic-bound, it is scarified to a depth of 2 or 3 in. Then it is treated with just an ordinary fuel oil in two or three applications, the harrows being run over it after each time. A blade machine is used to move the material back and forth on the road until it is thoroughly mixed, as indicated by the color of the mass. After mixing in this manner the loose material is spread uniformly over the width to be surfaced and the road is opened to traffic, by which it is consolidated.

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* Chf., Div. of Constr., U. S. Bureau of Public Roads, Washington, D. C.

This kind of surfacing costs from $800 to $1 500 per mile for the oil and mixing process, and it is very effective. The writer has seen some sections that appeared as if they were paved with bituminous concrete.

W. W. Crosby,* M. Am. Soc. C. E. (by letter).—For some years the writer has been protesting against a dogmatic assertion, to the effect that "the return in the form of economic transportation is the sole measure of justification for road improvement". This idea has been more or less widely spread and apparently is supported by National organizations the endorsements of which are expected to carry great weight.

The paper emphasizes the fact that other considerations beside relative economics are important in determining highway improvements. It is commended, therefore, to the careful attention of all highway engineers.

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Paper No. 1755

USE OF WATER ON FEDERAL IRRIGATION PROJECTS*

By E. B. Debler,† M. Am. Soc. C. E.

With Discussion by Messrs. Robert A. Monroe, Ivan E. Houk, George N. Carter, Ralph L. Parshall, and E. B. Debler.

Synopsis

From the initiation of each of the irrigation projects of the United States Bureau of Reclamation it has been customary to ascertain and record the use of water. Operation and maintenance charges have always been based on the quantity of water delivered to individual farms, a method which was made mandatory by Act of Congress approved August 13, 1914. Other justifications for the measurement of water have been: The desirability of securing data for use in designing distribution systems; the necessity for obtaining data to use in determining project areas; and the desire to foster the most economical use of the available irrigation supply.

The results of field measurements of water delivered to farms are assembled annually in the project offices and are summarized in their respective project histories.

The project histories, together with supplemental data otherwise obtained, constitute the source of information from which the data incorporated in this paper have been compiled. In abstracting the results of the field measurements many of the original compilations were revised so that comparable data for all projects could be presented herein.

In tabular form are given the quantities of water delivered to the farms on the various projects each month during the different years of operation, the data being arranged chronologically and one table being devoted to each

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Note.—The subject of this paper is one of several selected by the Special Committee on Irrigation Hydraulics for study and research. The paper was prepared for the consideration of the Committee in order to make available for general use much valuable data heretofore found only in the files of the United States Bureau of Reclamation.

* Published in March, 1929, *Proceedings*.

† Engr., U. S. Bureau of Reclamation, Denver, Colo.

project or each important sub-division thereof. The tables also include pertinent related data, such as total irrigated areas, total mileage of canals and laterals operated, precipitation during the irrigation season, mean air temperatures, percentages of irrigated areas in different crops, canal and lateral losses, waste, etc. A summary gives the average use of water on the various projects during the 10-year period from 1917 to 1926, inclusive. Pertinent related data and information are also shown, as in the more detailed tabulations.

It is believed that the data included in the tables will be found useful to most engineers engaged in irrigation work. The results of the field measurements of use of water cover a wide range in climatic and soil conditions as well as several years of project operation under varying stages of development. The tabulated data do not possess a high degree of scientific accuracy. Nevertheless, they are believed to be sufficiently accurate for the purposes for which they were obtained.

From the hitherto unpublished records of the U. S. Bureau of Reclamation, considerable data have been assembled and tabulated, to show the variation in use of irrigation water. The many projects included are indicated on Fig. 1, a map of the western part of the United States showing the locations of the Federal irrigation projects. Numbers enclosed in circles correspond to the numbers of the tables containing the detailed data on the use of water.

Compilations for these twenty-four projects are given in Tables 1 to 25, inclusive, corresponding. The data selected are those considered most useful to an engineer in designing or reviewing an irrigation plan. In general, the information is self-explanatory and hence many of the tables require no comment. However, a few remarks may well be given dealing with the particular aspects of the general problem and of this special investigation.

METHODS OF MEASUREMENT

Reliable gauging stations are maintained at the diversion points on all projects. Discharge measurements at the heads of laterals are often complicated by canal structures. Waste, as the term is used in this compilation, is the water returned to natural streams through wasteways operated in connection with the project system. It does not include water delivered to the user and by him permitted to leave his farm unused. Project waste is measured by the project operation forces. The data on waste and losses are made to portray conditions without internal reservoirs by eliminating the reservoir losses where such reservoirs are present.

Farm deliveries are ascertained in a number of ways and with varying degrees of accuracy. In general, they are measured at, or very close to, the farm border; but in a few cases complications attending such observations have resulted in a practice of making measurements nearer the main canals with allowance for losses in transit to the farm border. The most common form of measurement is by means of the Cipolletti weir. However, a lack of adequate heads often precludes this method, so that the use of submerged weirs or current meters becomes necessary. The cost of current-meter work precludes the securing of accurate delivery records where this method is required. The use of the current-meter method is practically limited to parts of the Rio Grande and Yuma Projects.

In addition to the difficulties of water measurement, the procurement of records truly reflecting the use of water is complicated on many projects by human factors, in that the ditch rider is constantly besieged by individuals to reduce their water charges. On many projects there is, therefore, a recognized factor of under-measurement of farm deliveries, with a corresponding increase in apparent canal and lateral losses, because diverted waters not accounted for in waste and deliveries, are charged to canal loss. At numerous times and places, measurements have been made to ascertain the loss in feet of depth per day for various types of canal materials, but the utilization of these results for an estimate of total canal losses in an operating system is impractical, by reason of cost and changing conditions.

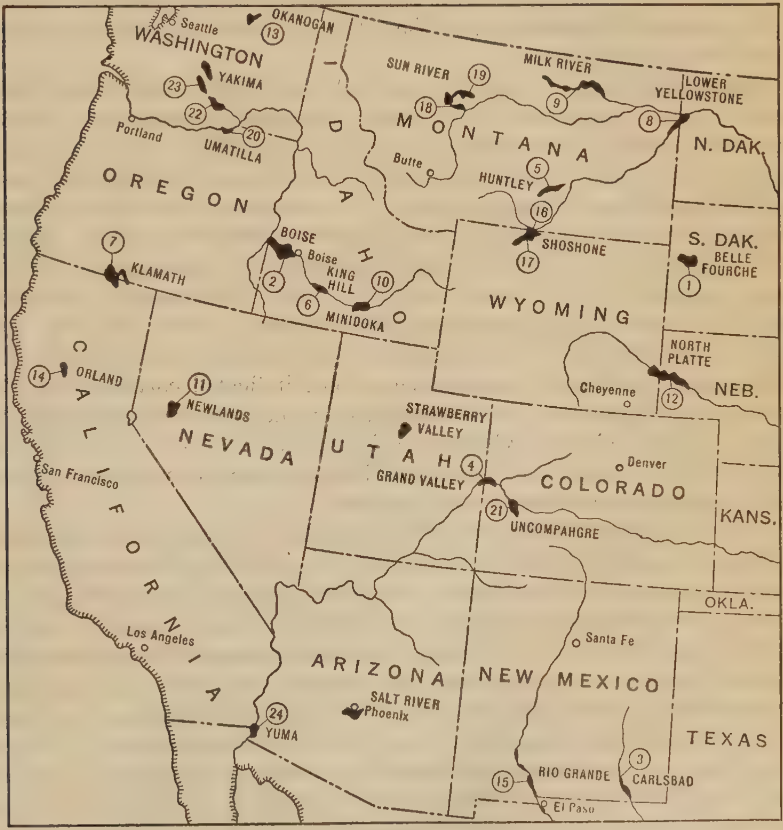
Seattle
WASHINGTON
OKANOGAN
13
YAKIMA
23
22
20
Portland
UMATILLA
OREGON
IDAHO
SUN RIVER
19
18
MONTANA
Butte
MILK RIVER
9
LOWER YELLOWSTONE
8
N. DAK.
HUNTLEY
5
16
SHOSHONE
17
S. DAK.
BELLE FOURCHE
1
BOISE
Boise
KING HILL
2
10
6
MINIDOKA
7
KLAMATH
WYOMING
NORTH PLATTE
12
NEB.
Cheyenne
14
ORLAND
CALIFORNIA
11
NEWLANDS
NEVADA
UTAH
STRAWBERRY VALLEY
4
GRAND VALLEY
COLORADO
Denver
21
UNCOMPAHGRE
KANS.
San Francisco
OKLA.
Santa Fe
Los Angeles
ARIZONA
NEW MEXICO
SALT RIVER
Phoenix
24
YUMA
TEXAS
3
CARLSBAD
15
RIO GRANDE
El Paso

FIG. 1.

MEANING OF THE DATA

Referring to the tabulations for the individual projects, the area commanded by the constructed canal system represents the irrigable area to which water could have been supplied. The irrigable area, with minor deviations on various projects, excludes rights of way for section-line roads, highways, railroads, main canals, principal laterals, and drains. It also excludes all lands that cannot be cultivated at moderate costs for land preparation and farm lateral construction, or lands that are unworthy of cultivation by reason of inferior soils. House and barn yards have not been excluded. Areas that are temporarily non-irrigable by reason of seepage are included. Declining areas are usually due to elimination of lands on account of inferior agricultural values or on account of permanent seepage. The lag in the irrigated area is due to delayed settlement and delayed development of individual holdings.

The comparison of irrigated and commanded lands, together with data on the character of canal construction and length of canals, is given as a guide for the interpretation of canal losses and project waste which can usually be expected to decline as full development is approached. In many cases the main canals have been constructed to a capacity that is considered adequate for the ultimate irrigable area, so that their operation to partial capacity is productive of undue losses.

The farm deliveries represent, in general, the quantities for which charges are made. On a few projects, "off-season" deliveries are customarily made to supply stock water and on others the use of "off-season" water is at times encouraged for leaching purposes and for conserving the limited supplies available during the irrigation season. The general practice is to make a small charge, or no charge at all, for "off-season" deliveries. The tabulation excludes such deliveries and the attendant losses, in so far as possible without research into the original water records. The column headed, "Actual Delivery Percentage Larger", represents the required increase necessary in the tabulated farm deliveries to ascertain actual deliveries, on account of the recognized factor of under-measurement. The indicated correction factor to ascertain actual deliveries has been supplied by the operating personnel. This is necessarily an opinion only, but it is supported by long experience and familiarity with project conditions, and in most cases by numerous although discontinuous measurements for the purpose.

Data on temperature, precipitation, character of crops, and character of soils, have been added to aid in interpreting the results. The growing season mentioned comprises the months in which, if rainfall were inadequate, it would be necessary to resort to irrigation. As a further guide, the precipitation during the non-growing months preceding the growing season also has been shown, because in many cases it replenishes soil moisture used in the growing season.

Alfalfa and forage crops, together with irrigated pastures, are heavily irrigated throughout the season—as a rule, by surface flooding. Fields of small grains, although usually irrigated by flooding, require the least total water because the growing period during the irrigation season is short and plant requirements are relatively small. Furrow crops, of which potatoes, sugar-beets, corn, and small fruits are predominant, require more water than small grains, but generally less than the forage crops. Trees, usually fruit orchards, have a low water requirement except when intercropped and then such requirement is largely determined by the character of the intercropping.

TABLE 1.—USE OF WATER ON THE BELLE FOURCHE IRRIGATION PROJECT OF SOUTH DAKOTA.

(Average elevation, 2 800 ft.; in clay and sandy soil; 58 miles concrete-lined.)

Calendar year.	Irrigated area, total for year.[2]	Area commanded by constructed canal system.[8]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[3]													Precipitation, May to September, in feet.[1]	Water delivered plus precipitation, May to September, in feet.	Precipitation, October to April, preceding, in feet.[1]	Mean temperature, May to September, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.			PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Canal and lateral losses.[4]	Waste.	Delivered to farms.[7]
1912	27 897	65 852	448	......	......	......	......	0.00	0.68	0.27	0.03	0.11	......	......	......	1.09	1.10	2.19	0.50	63	25	61	14	32	16	52
1913	32 881	65 868	462	......	......	......	......	0.02	0.39	0.69	0.23	0.11	......	......	......	1.40	0.76	2.20	0.25	66	31	59	10	51[5]	[5]	49
1914	37 454	68 852	498	......	......	......	......	0.02	0.48	0.57	0.24	0.14	......	......	......	1.45	0.78	2.23	0.45	67	43	43	14	44	7	49
1915	44 067	78 591	528	......	......	......	......	0.01	0.02	0.10	0.13	0.09	0.02	......	......	0.37	1.49	1.86	0.56	60	52	32	16	31	35	34
1916	48 468	78 567	529	......	......	......	......	0.01	0.09	0.47	0.10	0.13	0.01	......	......	0.81	0.86	1.67	0.32	61	59	31	10	16[6]	59	25
1917	50 272	83 335	612	......	......	......	......	0.00	0.19	0.57	0.30	0.15	......	......	......	1.21	0.49	1.70	0.47	63	60	27	13	31	15	54
1918	52 445	82 592	612	......	......	......	......	0.03	0.46	0.17	0.26	0.07	......	......	......	0.99	1.04	2.03	0.48	66	60	30	10	34	18	48
1919	56 638	82 797	615	......	......	......	......	0.18	0.51	0.45	0.23	0.09	......	......	......	1.46	0.33	1.79	0.56	67	63	28	9	35	11	54
1920	43 022	82 655	615	......	......	......	......	0.00	0.00	0.14	0.48	0.13	......	......	......	0.75	1.39	2.14	0.74	67	63	28	9	42	6	52
1921	55 100	82 363	615	......	......	......	0.01	0.16	0.29	0.44	0.26	0.13	......	......	......	1.29	0.63	1.92	0.29	67	64	23	13	38	11	51
1922	31 150	82 194	545	......	......	......	......	0.01	0.01	0.08	0.50	0.31	......	......	......	0.91	1.19	2.10	0.42	66	65	21	14	37	16	47
1923	30 552	81 843	506	......	......	......	......	0.06	0.14	0.36	0.07	0.10	......	......	......	0.73	1.68	2.41	0.40	65	67	14	19	39	26	35
1924	48 400	81 897	455	......	......	......	......	0.02	0.38	0.37	0.30	0.13	......	......	......	1.20	0.46	1.66	0.48	62	67	9	23	38	19	43
1925	48 800	81 816	455	......	......	......	......	0.12	0.16	0.52	0.38	0.12	......	......	......	1.30	0.57	1.87	0.64	65	59	17	24	42	13	45
1926	36 260	74 569	443	......	......	......	......	0.05	0.11	0.48	0.39	0.15	......	......	......	1.18	0.87	2.05	0.34	60	54	25	21	39	23	38

[1] Climatological records, mean of Orman and Vale, S. Dak.

[2] 1912–19, total cropped area; irrigated area considerably less in years of high rainfall.

[3] Project charges based on these deliveries; actual deliveries estimated as 15% greater.

[4] Actual losses estimated as averaging 7% less than shown, due to under-measurement of deliveries.

[5] Waste not measured, but included in loss.

[6] 49% of total diversions wasted in order to lower reservoir water surface for dam repairs.

[7] Actual deliveries estimated as averaging 7% more than shown, due to under-measurement.

[8] Irrigable area originally estimated at 90 000 acres; reduction due to suspension of construction and to elimination of poor lands.

TABLE 2.—USE OF WATER ON THE BOISE IRRIGATION PROJECT OF IDAHO.

(Average elevation, 2 500 ft.; chiefly in volcanic ash and sandy loam soil; 37 miles of pipe, flume, and concrete-lined canals.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[9]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[4]													Precipitation, April to October, in feet.	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.	Mean temperature, April to October, in degrees Fahrenheit.	PERCENTAGE OF AREA IN DIFFERENT CROPS.[1]				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.[5]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.[6]	Waste.[7]	Delivered to farm.[8]
1912	61 725	120 000	966	......	......	......	0.02	0.26	0.55	0.61	0.26	0.17	0.02	......	[3]	1.89	0.77	2.66	0.58	59	48	39	7	6	51	4	45
1913	76 265	127 000	969	......	......	......	0.04	0.57	0.72	0.54	0.36	0.12	0.03	......	[3]	2.38	0.78	3.16	0.30	60	48	38	9	5	....	....	....
1914	83 714	127 000	970	......	......	......	0.29	0.70	0.57	0.61	0.27	0.12	0.06	......	[3]	2.62	0.44	3.06	0.43	62	52	28	15	5	40	4	56
1915	97 127	146 190	979	......	......	......	0.37	0.19	0.73	0.68	0.55	0.25	0.04	......	......	2.81	0.40	3.21	0.29	61	47	36	13	4	37	2	61
1916	101 315	143 810	981	......	......	......	0.26	0.69	0.62	0.89	0.64	0.39	0.07	......	......	3.56	0.32	3.88	0.49	60	55	34	7	4	35	6	59
1917	116 686	143 780	988	......	......	......	....	0.37	0.81	0.76	0.64	0.47	0.02	......	......	3.07	0.36	3.43	0.35	61	55	34	6	5	36	3	61
1918	131 500	163 938	989	......	......	......	0.15	0.86	0.83	0.90	0.65	0.34	0.02	......	......	3.75	0.39	4.14	0.48	63	54	36	7	3	28	6	66
1919	146 500	165 305	989	......	......	......	0.12	0.89	0.92	0.89	0.50	0.02	....	......	......	3.34	0.16	3.50	0.38	62	55	32	10	3	29	1	70
1920	150 760	165 841	1 002	......	......	......	0.11	0.80	0.85	0.95	0.77	0.10	0.02	.....	......	3.60	0.43	4.03	0.42	61	61	26	11	2	26	1	73
1921	153 000	167 483	1 016	......	......	......	0.25	0.53	0.79	0.90	0.75	0.33	0.09	......	......	3.64	0.41	4.05	0.55	62	57	30	11	2	26	1	74
1922	155 000	167 555	1 016	......	.....	......	0.04	0.71	0.81	0.91	0.70	0.26	0.04	......	......	3.47	0.28	3.75	0.40	62	48	30	20	2	29	....	71
1923	155 500	167 655	1 019	......	......	......	0.37	0.73	0.43	0.90	0.77	0.45	0.05	......	......	3.70	0.65	4.35	0.29	61	49	29	19	3	32	....	68
1924	156 000	167 754	1 019	(Severe shortage)			0.18	0.72	0.50	0.25	0.04	0.02	0.01	0.05	......	1.77	0.18	1.95	0.25	62	61	22	14	3	36	....	64
1925	156 000	167 782	1 019	......	.....	......	0.27	0.93	0.78	0.88	0.91	0.33	0.05	......	......	4.15	0.40	4.55	0.55	62	47	33	18	2	21	....	79
1926	158 000	167 780	1 019	(Heavy shortage)			0.26	0.76	0.62	0.45	0.24	0.09	0.01	......	......	2.43	0.29	2.72	0.44	64	58	31	8	3	32	....	68

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[1] Based on crop census for an average of 75% of irrigated area.

[2] Climatological records at Caldwell, Idaho.

[3] Arrowrock Reservoir under construction, placed in operation in 1915, with supply to lands above Deerflat Reservoir limited prior thereto.

[4] Project charges based on these deliveries; actual deliveries estimated as 5% greater.

[5] Total diversions from Boise River at Boise Diversion Dam above Boise, Idaho, plus outflow from Deerflat Reservoir minus inflow to Deerflat Reservoir except for 1912 when there was other inflow to the reservoir; Deerflat Reservoir is served by main canal.

[6] Actual losses estimated as averaging 3% less than shown due to under-measurement of deliveries.

[7] Includes only wasteways where records were kept. No records after 1922. Waters wasted from upper system but caught in Deerflat Reservoir not charged as waste.

[8] Actual deliveries estimated as averaging 3% more than shown due to under-measurement.

[9] Final irrigable area 167 780 acres (under Government canals, with water supply from Boise River).

TABLE 3.—USE OF WATER ON THE CARLSBAD IRRIGATION PROJECT OF NEW MEXICO.

(Average elevation, 3 100 ft.; in sandy loam and gravel; 25% of system concrete-lined.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canal system.[6]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.													Precipitation, February to November, in feet.[1]	Water delivered plus precipitation, February to November, in feet.	Precipitation, December to January preceding, in feet.[1]	Mean temperature, February to November, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.[3]		
				January.[7]	February.[7]	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.[8]	Trees.	Canal and lateral losses.[4]	Waste.[9]	Delivered to farms.[5]
1912	13 459	20 277	45	......	......	0.12	0.45	0.44	0.48	0.44	0.62	0.18	0.10	0.05	......	2.88	0.99	3.87	0.08	65	63	....	35	2	48	6	46
1913	14 260	20 261	45	......	......	0.06	0.49	0.43	0.17	0.44	0.40	0.21	0.06	0.06	......	2.32	1.20	3.52	0.08	66	70	1	26	3	54	7	39
1914	12 690	20 261	45	......	0.05	0.05	0.52	0.32	0.17	0.50	0.55	0.26	0.02	......	......	2.44	1.31	3.75	0.07	67	71	....	27	2	56	9	35
1915	13 470	24 796	45	......	......	......	0.06	0.32	0.58	0.43	0.35	0.21	0.15	0.04	......	2.14	1.46	3.60	0.30	66	54	15	29	2	49	15	36
1916	16 600	24 775	45	0.01	0.15	0.22	0.39	0.51	0.50	0.30	0.02	0.16	0.06	0.11	......	2.43	1.62	4.05	0.07	67	54	19	26	1	58	3	39
1917	16 882	24 775	45	0.03	0.02	0.15	0.53	0.42	0.21	0.08	0.40	0.21	0.19	0.10	......	2.34	0.46	2.80	0.03	66	53	13	34	...	52	1	47
1918	19 460	24 775	45	......	0.11	0.12	0.50	0.38	0.39	0.14	0.32	0.38	0.02	0.07	......	2.43	0.57	3.00	0.05	65	50	8	42	...	50	1	49
1919	20 363	24 991	45	......	0.02	......	0.43	0.37	0.41	0.46	0.49	0.21	0.01	......	......	2.40	1.56	3.96	0.05	65	44	3	53	...	53	4	43
1920	22 172	24 991	45	......	......	0.09	0.62	0.31	0.16	0.56	0.32	0.32	0.04	....	......	2.42	1.13	3.55	0.11	65	32	2	66	...	52	8	40
1921	23 814	24 991	45	......	0.10	0.17	0.52	0.21	0.09	0.47	0.51	0.27	0.12	0.03	.....	2.49	0.77	3.26	0.07	67	38	5	57	...	42	15	43
1922	24 078	24 991	45	0.10	0.03	0.14	0.42	0.23	0.31	0.51	0.44	0.08	0.10	......	......	2.36	0.91	3.27	0.02	65	33	2	65	...	42	9	49
1923	24 063	24 991	45	0.13	0.02	0.01	0.55	0.39	0.35	0.41	0.36	0.14	0.02	......	......	2.38	1.02	3.40	0.04	65	25	1	74	...	47	5	48
1924	24 460	25 045	45	......	0.01	0.03	0.57	0.32	0.47	0.52	0.63	0.12	0.04	......	......	2.71	0.21	2.92	0.18	66	20	....	80	...	45	5	50
1925	24 778	25 045	45	0.12	0.14	0.10	0.43	0.11	0.07	0.17	0.51	0.13	0.06	......	......	1.84	0.80	2.64	0.04	68	19	....	81	...	49	8	43
1926	25 278	25 056	45	0.04	0.18	0.10	0.48	0.12	0.37	0.40	0.43	0.13	0.00	......	......	2.25	1.12	3.37	0.06	66	20	1	79	...	51	6	43

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[1] Climatological records at Carlsbad, N. Mex.

[2] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[3] Total diversion at Avalon is estimated by adding 5% to sum of measured discharges at Main Canal flume over Pecos River and East Canal Head-Gate. Diversions from Black River also included.

[4] Actual losses estimated as averaging 4% less than shown, due to under-measurement of deliveries.

[5] Actual deliveries estimated as averaging 4% more than shown, due to under-measurement.

[6] The project that may permanently be irrigated is limited by water supply and in years of abundant run-off may be exceeded as additional arable lands are commanded by the existing distribution system. The final irrigable area is 25 000 acres.

[7] Irrigation in these months is largely done in anticipation of shortages later in season and with waters that would otherwise be wasted.

[8] Largely cotton.

[9] Indicated waste is largely at ends of season and of waters which can not be conserved.

TABLE 4.—USE OF WATER ON THE GRAND VALLEY IRRIGATION PROJECT OF COLORADO.

(Average elevation, 4 700 ft.; much of system specially puddled-lined; main canal largely in highly pervious strata; many of laterals in sand, with porous subsoil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[9]	Miles of canals and laterals operated.[9]	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.													Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.[1]	Mean temperature, April to October, in degrees, Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.[8]		
				January.	February.[3]	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Canal and lateral losses.[5] [7]	Waste.[6]	Delivered to farm.[8]
1916	1 742	15 000	80	......	......	....	......	0.46	0.74	0.62	0.21	0.27	0.08	0.04	......	2.42	0.57	2.99	0.35	64	14	39	14	33	54	32	14
1917	5 289	30 500	137	......	......	......	0.06	0.25	0.70	0.97	0.72	0.38	0.22	0.21	0.03	3.54	0.39	3.93	0.15	64	20	30	35	15	44	23	33
1918	8 102	30 500	168	......	....	......	0.13	0.75	0.83	0.66	0.53	0.39	0.27	0.12	......	3.68	0.39	4.07	0.24	66	23	27	40	10	43	17	40
1919	10 049	30 500	175	......	......	......	0.10	0.63	0.95	0.82	0.68	0.42	0.21	......	......	3.81	0.35	4.16	0.24	67	29	25	42	4	44	17	39
1920	11 734	30 500	187	.....	......	......	......	0.46	0.68	0.69	0.63	0.33	0.28	......	......	3.07	0.45	3.52	0.45	64	38	25	34	3	49	18	33
1921	12 290	30 500	200	......	......	......	0.22	0.76	0.73	0.84	0.42	0.33	0.28	......	......	3.58	0.72	4.30	0.16	65	41	25	32	2	38	22	40
1922	12 372	30 500	212	......	.....	......	0.09	0.77	0.88	0.85	0.59	0.36	0.20	......	......	3.74	0.33	4.07	0.25	66	41	21	36	2	44	22	34
1923	12 870	30 500	212	......	......	......	0.16	0.79	0.75	0.82	0.70	0.42	0.13	......	......	3.77	0.44	4.21	0.32	64	49	11	38	2	42	24	34
1924	13 456	30 000	214	......	.....	......	0.25	0.96	0.68	0.97	0.92	0.38	0.18	......	......	4.34	0.54	4.88	0.27	65	49	21	27	3	39	20	41
1925	13 488	30 350	214	......	......	......	0.35	1.11	0.89	0.82	0.73	0.24	0.07	......	......	4.21	0.61	4.82	0.31	66	51	12	36	1	35	22	43

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[1] Climatological records at Grand Junction, Colo.

[2] Project lands only; Palisade, Mesa Company, and Orchard Mesa Irrigation Districts not included.

[3] Project charges based on these deliveries; actual deliveries estimated as 5% greater.

[4] Discharge of main canal at head less deliveries to Palisade, Mesa County, and Orchard Mesa Irrigation Districts.

[5] Includes main canal loss on deliveries to Districts in 1919 and subsequent years, carried a distance of about 5 miles in canal which is largely lined.

[6] Water supply, except on rare occasions, far in excess of project requirements.

[7] Actual losses estimated as averaging 3% less than shown, due to under-measurement of deliveries.

[8] Actual deliveries estimated as averaging 3% more than shown, due to under-measurement.

[9] Final irrigable area—45 000 acres—will require material extension of lateral system.

TABLE 5.—USE OF WATER ON THE HUNTLEY IRRIGATION PROJECT OF MONTANA.

(Average elevation, 3 000 ft.; practically all in earth.)

Calendar year.	Irrigated area, total for year.[8]	Area commanded by constructed canal system.[8]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.													Precipitation, May to September, in feet.[1]	Water delivered plus precipitation, May to September, in feet.	Precipitation, October to April preceding, in feet.[1]	Mean temperature, May to September, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.			PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.[3]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Canal and lateral losses.[4]	Waste.[3]	Delivered to farms.[7]
1912	14 425	28 805	194	......	......	......	......	0.06	0.38	0.36	0.57	0.11	......	......	......	1.48	0.92	2.40	0.45	61	30	25	45	17	42	41
1913	15 798	28 805	194	......	......	......	......	0.09	0.26	0.64	0.44	0.10	......	......	......	1.53	0.62	2.15	0.47	63	33	34	33	14	42	44
1914	17 068	28 805	194	......	......	......	......	0.01	0.29	0.61	0.47	0.05	......	......	......	1.43	0.76	2.19	0.45	63	38	31	31	19	43	38
1915	18 203	30 813	212	......	......	......	0.03	0.16	0.01	0.26	0.40	0.11	......	......	......	0.97	1.18	2.15	0.21	60	33	32	35	26	40	34
1916	18 635	32 936	232	......	......	......	......	0.09	0.19	0.45	0.29	0.10	......	......	......	1.12	0.61	1.73	0.27	63	39	30	31	27	42	31
1917	19 122	31 892	232	......	......	......	......	0.00	0.08	0.60	0.36	0.07	......	......	......	1.11	0.65	1.76	0.44	63	42	36	22	33	34	33
1918	18 958	32 884	232	......	......	......	......	0.02	0.40	0.31	0.28	0.05	......	......	......	1.06	0.66	1.72	0.38	66	43	44	13	38[5]	32[6]	30
1919	19 310	32 885	232	......	......	......	......	0.39	0.53	0.44	0.27	0.02	......	......	......	1.65	0.56	2.21	......	64	46	45	9	42	24	34
1920	20 020	32 824	232	......	......	......	......	0.00	0.28	0.45	0.38	0.10	......	......	......	1.21	0.53	1.74	0.45	64	49	36	15	31	38	31
1921	18 800	31 632	232	......	......	......	......	0.00	0.32	0.59	0.40	0.11	......	......	......	1.42	0.57	1.99	0.22	64	39	41	20	28	38	34
1922	19 523	32 000	232	......	......	......	......	0.00	0.03	0.50	0.28	0.15	......	......	......	0.96	0.72	1.68	0.47	65	41	38	21	37	37	26
1923	20 000	32 000	232	......	......	......	......	0.02	0.26	0.44	0.19	0.10	......	......	......	1.01	1.06	2.07	0.25	65	42	24	34	50	22	28
1924	19 600	32 540	232	......	......	......	......	0.07	0.12	0.42	0.47	0.18	......	......	......	1.26	0.50	1.76	0.36	63	43	17	40	38	32	30
1925	18 939	32 540	232	......	......	......	......	0.28	0.13	0.53	0.43	0.13	......	......	......	1.50	0.44	1.94	0.39	65	34	30	36	45	20	35
1926	19 790	32 540	232	......	......	......	0.01	0.30	0.35	0.36	0.36	0.07	......	......	......	1.45	0.59	2.04	0.33	65	40	27	33	48	22	30

[1] Climatological records at Huntley, 1912–18; at Ballantine, thereafter.

[2] Project charges based on recorded deliveries; actual deliveries estimated as 15% greater.

[3] Includes diversions for operation of pumps.

[4] Actual losses estimated as averaging 5% less than shown, due to under-measurement of deliveries.

[5] Includes all waste except diversions for operation of pumps.

[6] Diversions for operation of pumps only, no waste included.

[7] Actual deliveries estimated as averaging 5% more than shown, due to under-measurement.

[8] Final irrigable area estimated at about 32 000 acres, contains much adobe land which may prove infeasible of agricultural development.

TABLE 6.—USE OF WATER ON THE KING HILL IRRIGATION PROJECT OF IDAHO.

(Average elevation, 2 750 ft.; in volcanic ash and sandy soil; 40% concrete-lined or structures.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[6]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.													Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.	Waste.[4]	Delivered to farms.
1920	4 780	11 340	......	......	......	......	......	......	......	......	......	......	......	......	[2]	4.69	0.19	4.88	0.29	60	77	10	7	6	....	....	....
1921	5 908	13 648	90	......	......	......	[3]	0.40	1.01	1.46	1.25	0.98	......	......	[3]	5.08	0.42	5.50	0.46	66	78	5	8	9	39	8	53
1922	6 440	13 648	90	......	......	......	0.02	0.89	1.24	1.33	1.34	0.75	......	.....	......	5.57	0.31	5.88	0.52	65	74	6	11	9	39	2	59
1923	7 017	16 888	100	......	......	......	0.15	1.31	0.85	1.37	1.23	0.95	0.11	......	......	5.97	0.58	6.55	0.22	64	78	7	9	6	29	26	45
1924	6 235	16 888	100	......	......	......	0.58	1.66	1.56	1.67	1.63	1.12	0.39	0.04	......	8.65	0.14	8.79	0.15	64	77	5	12	6	31	18	51
1925	6 624	16 888	100	......	......	......	0.19	1.29	1.14	1.42	1.46	1.01	0.28	......	......	6.79	0.55	7.34	0.60	63	76	9	10	5	30	23	47
1926	6 364	16 890	93	......	......	......	0.68	1.63	1.75	1.67	1.73	1.09	0.38	......	......	8.84	0.18	9.02	0.23	64	75	9	11	5	[5]	[5]	58
1927	6 629	16 890	93	......	......	......	0.23	1.39	1.55	1.65	1.72	0.92	0.27	......	......	7.73	0.25	7.98	0.66	61	67	10	18	5	[5]	[5]	54

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[1] Climatological records, Glenns Ferry, Idaho.

[2] Project operated prior to 1921 by King Hill Irrigation District, which again took over operation in January, 1927.

[3] Unrecorded diversions and deliveries, April 4 to May 18, 1921, prior to operation by U. S. Reclamation Bureau, not included.

[4] Project has a fixed uniform supply of available water, waste occurring largely at ends of season of waters that could not be conserved.

[5] Loss and waste not segregated in 1926 and 1927.

[6] Irrigable area, 16 890 acres, but will probably remain to a large degree unirrigated on account of unexpectedly heavy deliveries required.

TABLE 7.—USE OF WATER ON THE KLAMATH IRRIGATION PROJECT OF OREGON AND CALIFORNIA.

(Average elevation, 4 100 ft.; chiefly in sandy loam soil; 2 miles concrete-lined; Langell Valley areas not included.)

Calendar year.	Irrigated area, total for year.[2]	Area commanded by constructed canals.[13]	Miles of canals and laterals operated.[4]	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[5]													Precipitation, May to September, in feet.[1]	Water delivered plus precipitation, May to September, in feet.	Precipitation, May to September preceding, in feet.[1]	Mean temperature, May to September, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.[6]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.[7] [9]	Waste.	Delivered to farms.[1]
1912	23 834	30 093	132	......	......	......	......	0.17	0.29	0.42	0.21	0.04	......	......	......	1.13	0.60	1.73	0.94	57	59	38	3	...	36	[8]	....
1913	18 928	28 900	146	......	......	......	......	0.26	0.43	0.23	0.21	0.04	......	......	......	1.17	0.40	1.57	0.71	60	66	32	2	...	.....	[8]	....
1914	24 440	38 000	205	......	......	......	......	0.31	0.29	0.39	0.21	0.06	......	......	......	1.26	0.23	1.49	0.76	61	65	33	2	...	40	3	57
1915	27 254	38 000	205	......	......	......	0.01	0.15	0.47	0.30	0.16	0.03	......	......	......	1.12	0.19	1.31	0.55	60	58	40	2	...	50	6	44
1916	29 351	45 272	216	......	......	......	0.01	0.21	0.38	0.14	0.24	0.04	......	......	......	1.02	0 31	1.33	0.62	59	58	41	1	...	49	6	45
1917	33 635	44 715	216	......	......	......	......	......	0.38	0.31	0.26	0.03	......	......	......	0.98	0.17	1.15	0.65	61	60	39	1	...	42	8	50
1918	38 268	50 000	210	......	......	......	0.02	0.38	0.39	0.29	0.24	0.04	......	......	[3]	1.36	0.19	1.55	0.51	60	69	30	1	...	40	10	50
1919	42 881	50 000	216	......	......	......	......	0.36	0.34	0.32	0.28	0.02	......	......	[3]	1.32	0.16	1.48	0.62	61	74	25	1	...	47	6	47
1920	44 800	50 000	216	......	......	......	......	0.28	0.30	0.21	0.29	0.03	......	......	[3]	1.11	0.18	1.29	0.65	59	76	23	1	...	55	2	43
1921	43 883	50 000	216	......	......	......	......	0.15	0.36	0.27	0.28	0.05	......	......	[3]	1.11	0.15	1.26	0.97	60	79	20	1	...	52	2	46
1922	44 929	51 023	216	......	......	......	......	0.11	0.39	0.28	0.27	0.06	......	......	[3]	1.11	0.19	1.30	0.84	63	79	19	2	...	52	6	42
1923	46 624	54 171	277	......	......	......	.. ...	0.31	0.30	0.22	0.31	0.07	......	......	[3]	1.21	0.23	1.44	0.77	59	82	16	2	...	24	30	46
1924	47 300	53 906	277	......	......	......	0.04	0.41	0.36	0.34	0.21	0.04	......	......	[4]	1.40	0.11	1.51	0.45	62	89	9	2	...	50[10]	9	41[10]
1925	47 400	55 167	277	......	......	......	......	0.26	0.33	0.35	0.23	0.02	......	......	[4]	1.19	0.40	1.59	0.84	59	82	15	3	...	48[10]	12	40[10]
1926	48 037	64 983	277	......	......	......	0.06	0.43	0.44	0.37	0.25	0.06	......	......	[11]	1.61	0.05	1.66	0.55	61	78	17	5	...	53[10]	6	41[10]

[1] Climatological records, mean of Klamath Falls and Merrill, Ore.

[2] Private districts included, 1918 and 1919, 5 000 acres; 1920, 6 700 acres; 1921, 7 800 acres; 1922, 1923, 1924, 8 929 acres; 1925, 8 498 acres; 1926, 9 883 acres.

[3] Deliveries to private districts at heads of laterals instead of farms.

[4] Private districts not included. Quantities shown are for project lands only.

[5] Project charges based on these quantities; actual deliveries estimated as 20% greater.

[6] Total diversions from Upper Klamath Lake, Lost River, and Lost River return flow.

[7] Losses in private district laterals not included (about 16% of irrigated area in private districts 1918-23).

[8] No record.

[9] Actual losses estimated as averaging 8% less than shown, due to under-measurement of deliveries.

[10] Based on an estimate of lateral loss on deliveries to private districts.

[11] Based on quantities delivered to private district lands as well as project lands.

[12] Actual deliveries estimated as averaging 8% more than shown, due to under-measurement.

[13] Final irrigable area, 95 000 acres, including lands in private projects served by project main canals.

TABLE 8.—USE OF WATER ON THE LOWER YELLOWSTONE IRRIGATION PROJECT OF MONTANA.

(Average elevation, 1 900 ft.; all in earth.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[7]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[3]													Precipitation, May to September, in feet.[1]	Water delivered plus precipitation, May to September, in feet.	Precipitation, October to April, preceding, in feet.[1]	Mean temperature, May to September, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.			Percentage of Total Diversions for Season.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Canal and lateral losses.[5]	Waste.[4]	Delivered to farms.[6]
1912	5 068	37 880	126	......	......	......	......	0.00	0.08	0.98	0.11	0.01	0.01	......	......	1 19	1.05	2.24	0.56	58	12	87[2]	1	43	18	39
1913	7 660	36 578	133	......	......	......	......	0.00	0.71	0.46	0.09	0.06	0.02	......	......	1.34	0.64	1.98	0.28	63	23	75	2	52	14	34
1914	5 743	36 250	151	......	....	......	......	0.04	0.33	0.66	0.46	0.04	0.06	......	......	1.59	0.93	2.52	0.25	64	55	39	6	....	....	....
1915	12 656	42 329	163	......	......	......	......	0.20	0.35	0.67	0.19	0.01	......	......	......	1.42	1.09	2.51	0.23	59	42	56	2	43	12	45
1916	6 020	42 288	151	......	......	......	......	0.00	0.22	0.68	0.24	0.11	......	......	......	1.25	1.22	2.47	0.44	62	58	40	2	48	24	28
1917	15 744	42 232	180	......	......	......	......	0.02	0.39	0.78	0.48	0.10	......	......	......	1.77	0.35	2.12	0.43	63	50	42	8	43	11	46
1918	21 075	42 272	186	......	......	......	......	0.03	0.47	0.55	0.06	0.00	......	......	......	1.11	0.72	1.83	0.36	64	40	53	7	47	8	45
1919	21 300	42 167	188	......	......	......	......	0.00	0.38	0.56	0.21	0.08	......	......	......	1.23	0.45	1.68	0.36	66	52	43	5	57	5	38
1920	19 120	42 167	187	......	......	......	......	0.00	0.34	0.36	0.11	0.06	......	......	......	0.87	0.63	1.50	0.47	65	43	49	8	50	15	35
1921	19 980	40 200	174	......	......	......	......	0.00	0.53	0.35	0.35	0.06	......	......	......	1.29	0.53	1.82	0.37	64	51	35	14	44	16	40
1922	15 599	40 200	155	......	......	......	......	0.00	0.22	0.57	0.33	0.06	......	......	......	1.18	0.83	2.01	0.14	66	48	31	21	40	23	37
1923	17 857	58 000	268	......	......	......	......	0.00	0.22	0.55	0.32	0.17	......	......	......	1.26	0.96	2.22	0.28	66	42	27	31	41	34	25
1924	14 025	58 000	226	......	......	......	......	0.00	0.18	0.20	0.66	0 22	......	......	......	1.26	0.71	1.97	0.32	62	41	4	55	41	36	23
1925	18 276	58 243	243	......	......	......	......	0.24	0.18	0.40	0.60	0.17	0.03	......	......	1.62	0.75	2.37	0.34	64	35	10	55	42	31	27
1926	23 331	58 248	255	......	......	......	......	0.42	0.24	0.59	0.36	0.18	0.01	......	......	1.80	0.64	2.44	0.13	64	33	32	35	37	29	34

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[1] Climatological records at Savage, Mont.

[2] For entire cropped area of 17 500 acres.

[3] Project charges based on recorded deliveries; actual deliveries estimated as 5% greater.

[4] Large waste due to excess of supply over all project requirements and to silt-sluicing operations.

[5] Actual losses estimated as averaging 2% less than shown, due to under-measurement of deliveries.

[6] Actual deliveries estimated as averaging 2% more than shown, due to under-measurement.

[7] Final irrigable area, 59 349 acres.

TABLE 9.—USE OF WATER ON THE MILK RIVER IRRIGATION PROJECT OF MONTANA.

(Average elevation, 2 200 ft.; in earth, largely river bottom; Chinook Division not included.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[10]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[3]													Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March, preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.			Percentage of Total Diversions for Season.[4]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Canal and lateral losses.[5 6]	Waste.	Delivered to farm.[9]
1912	352	7 800	30	......	......	......	.....	......	......	0.62	0.11	0.05	0.05	......	......	0.83	1.38	2.21	0.15	......	10	89	1	....	....	...
1913	2 545	12 800	59	......	......	......	0.01	0.09	0.12	0.56	0.08	0.01	0.05	......	......	0.92	0.79	1.71	0.13	......	34	66	....	....	....	....
1914	2 201	13 440	53	......	......	......	0.14	0.40	0.06	0.20	......	......	......	......	......	0.80	1.15	1.95	0.11	57	44	55	1	41	6	53
1915	4 192[2]	22 200	86	......	......	.....	0.12	0.26	0.06	0.17	0.01	0.01	0.05	......	......	0.70	1.13	1.83	0.20	58	60	39	1	61[7]	16	23
1916	5 518[2]	40 400	184	.....	......	......	0.01	0.23	0.07	0.16	0.12	0.03	0 06	....	......	0.68	1.04	1.72	0.47	56	76	23	1	17	7	76
1917	11 058	45 000	204	......	......	......	......	0.11	0.34	0.40	0.09	0.01	0.03	0.03	......	1.01	0.44	1.45	0.24	57	65	33	2	25	10	65
1918	24 842	58 750	254	......	......	......	.....	0.05	0.33	0.27	0.02	0.01	......	......	......	0.68	0.73	1.41	0.19	59	68	31	1	32	9	59
1919	25 485	58 900	317	......	......	......	......	0.08	0.40	0.27	0.06	0.02	0.01	......	......	0.84	0.59	1.43	0.13	59	80	19	1	31	8	61
1920	18 500	68 600	361	......	......	......	......	0.06	0.25	0.16	0.09	0.01	......	......	......	0.57	1.12	1.69	0.13	58	78	21	1	36	26	38
1921	11 440	69 200	247	......	......	......	0.01	0.05	0.29	0.08	0.08	0.01	0.02	......	......	0.54	1.14	1.68	0.14	58	75	23	2	33	14[8]	53
1922	12 011	66 300	255	......	......	......	0.01	0.03	0.15	0.23	0.05	0.01	0.03	......	.....	0.51	0.94	1.45	0.25	59	81	16	3	26	10	64
1923	13 622	64 750	253	......	......	......	0.04	0.28	0.13	......	0.03	0.01	0.01	......	......	0.50	1.38	1.88	0.12	58	82	14	4	24	23	53
1924	14 594	65 300	284	.....	......	......	0.03	0.15	0.11	0.21	0.08	0.02	0.01	......	......	0.61	1.11	1.72	0.14	56	76	14	10	39	42	19
1925	19 892	64 755	284	......	......	......	......	0.10	0.08	0.26	0.11	0.02	......	......	......	0.57	0.93	1.50	0.28	56	72	13	15	52	31	17
1926	16 485	73 252	287	......	......	......	0.03	0.19	0.15	0.25	0.07	0.03	......	......	......	0.72	0.72	1.44	0.16	58	54	36	10	63	17	20

[1] Mean of Glasgow and Malta, Mont., climatological records, 1912–26.

[2] From letter of March 17, 1925, from Project Superintendent.

[3] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[4] Total diversions from Milk River plus canal diversions from Nelson Reservoir.

[5] Includes loss in Main Canal on diversions for Nelson Reservoir, but not loss in Nelson Reservoir.

[6] Actual losses estimated as averaging 5% less than shown, due to under-measurement of deliveries.

[7] Includes loss due to priming new canals and filling Nelson Reservoir below elevation of zero storage of 52.5 per cent.

[8] Includes waste to Bowdoin Lake and to Alkali Creek to facilitate river-bank protection work (6.9 per cent.).

[9] Actual deliveries estimated as averaging 5% more than shown, due to under-measurement.

[10] Final irrigable area, 145 100 acres.

TABLE 10.—USE OF WATER ON THE SOUTH SIDE PUMPING DIVISION OF THE MINIDOKA IRRIGATION PROJECT, IDAHO.

(Average elevation, 4 200 ft.; in sandy loam soil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[3]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.													Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.[2]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.	Waste.	Delivered to farm.
1912	....	....	......	......	......	......	......	0.12	0.47	0.58	0.28	0.20	0.01	......	......	1.66	0.83	2.49	0.49	55	30	48	20	2	....	....	....
1913	32 084	47 770	250	......	......	......	......	0.24	0.34	0.58	0.53	0.31	0.01	......	......	2.01	0.61	2.62	0.22	58	37	41	20	2	....	....	....
1914	35 788	47 770	257	......	......	......	......	0.42	0.44	0.70	0.51	0.22	......	......	......	2.29	0.70	2.99	0.62	60	48	35	16	1	37	7	56
1915	38 188	49 000	274	......	......	......	0.22	0.22	0.52	0.74	0.59	0.41	0.04	......	......	2.74	0.50	3.24	0.22	59	53	32	14	1	36	4	60
1916	40 643	49 000	275	......	......	......	0.02	0.38	0.59	0.66	0.60	0.40	0.02	......	......	2.67	0.43	3.10	0.74	57	57	28	14	1	37	4	59
1917	43 220	49 000	276	......	......	......	......	0.08	0.63	0.75	0.67	0.30	0.01	......	......	2.44	0.46	2.90	0.45	58	57	29	14	...	33	6	61
1918	44 765	48 962	276	......	......	......	0.02	0.53	0.59	0.66	0.53	0.28	0.02	......	......	2.63	0.41	3.04	0.34	60	52	34	14	...	39	3	58
1919	45 000	48 962	276	......	......	......	......	0.48	0.63	0.52	0.42	0.28	0.02	......	......	2.35	0.39	2.74	0.26	60	51	29	20	...	41	1	58
1920	46 133	48 962	276	......	......	......	0.03	0.44	0.65	0.62	0.60	0.32	0.02	......	......	2.68	0.54	3.22	0.21	58	51	27	22	...	38	5	57
1921	46 573	48 962	276	......	......	......	......	0.19	0.40	0.67	0.62	0.20	0.05	......	......	2.13	0.77	2.90	0.38	58	44	29	27	...	38	6	56
1922	45 286	48 960	274	......	......	......	......	0.18	0.59	0.70	0.52	0.34	0.05	......	......	2.38	0.63	3.01	0.47	60	42	27	31	...	42	4	54
1923	45 166	48 960	274	......	......	......	......	0.35	0.42	0.65	0.60	0.46	0.01	......	......	2.49	1.06	3.55	0.20	58	44	26	30	...	41	3	56
1924	43 404	48 960	274	......	......	......	0.23	0.63	0.49	0.59	0.43	0.11	......	......	......	2.48	0.28	2.76	0.23	60	52	20	28	...	39	0	61
1925	45 037	48 960	274	......	......	......	0.19	0.46	0.55	0.71	0.65	0.25	......	......	......	2.81	0.61	3.42	0.36	59	50	29	21	...	41	3	56
1926	44 875	48 960	274	......	......	......	0.17	0.71	0.68	0.69	0.55	0.19	......	......	......	2.99	0.17	3.16	0.26	61	55	29	16	...	35	0	65

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[1] Climatological records, 1912-17, Rupert; 1918-26, Burley, Idaho.

[2] Delivered by pumps at first lift. Does not include stock water in non-irrigation months, nor canal losses from Minidoka Dam to first lift, a distance of 14 miles.

[3] Final irrigable area, 48 960 acres.

TABLE 11.—USE OF WATER ON THE CARSON DIVISION OF THE NEWLANDS IRRIGATION PROJECT, NEVADA.

(Average elevation, 4 000 ft.; canals and laterals about 60% in sandy soil and 40% in clay.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[8]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[4]													Precipitation, March to October, in feet.[1]	Water delivered plus precipitation, March to October, in feet.	Precipitation, November to February, preceding, in feet.[1]	Mean temperature, March to October, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.[2]			PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September,	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Canal and lateral losses.[5] [6]	Waste.	Delivered to farms.[7]
1912	34 970	....	240	......	......	0.13	0.31	0.55	0.66	0.16	0.13	0.19	0.09	......	[3]	2.22	0.31	2.53	0.03	57	80	17	3	41	1	58
1913	40 818	....	240	......	......	0.19	0.50	0.64	0.26	0.24	0.19	0.17	0.11	......	[3]	2.30	0.52	3.80	0.04	59	84	10	6	38	6	56
1914	36 466	....	245	......	......	0.07	0.50	0.65	0.53	0.63	0.44	0.22	0.12	......	......	3.16	0.09	3.25	0.34	60	89	8	3	27	22	51
1915	36 853	....	250	......	......	0.05	0.31	0.39	0.57	0.54	0.37	0.20	0.06	0.05	......	2.54	0.34	2.88	0.13	59	83	12	5	30	24	46
1916	35 710	....	250	......	......	0.04	0.41	0.62	0.63	0.66	0.36	0.20	0.01	0.01	......	2.94	0.20	3.14	0.30	58	86	12	2	32	23	45
1917	36 350	....	294	......	......	0.01	0.21	0.53	0.61	0.63	0.39	0.25	0.07	......	......	2.70	0.09	2.79	0.14	59	83	9	8	35	23	42
1918	37 718	....	294	......	......	0.05	0.32	0.60	0.52	0.52	0.33	0.22	0.03	......	......	2.59	0.42	3.01	0.09	60	82	16	2	37	18	45
1919	39 547	....	300	......	......	0.05	0.31	0.56	0.50	0.56	0.40	0.30	0.03	......	......	2.71	0.18	2.89	0.07	60	84	14	2	46	14	40
1920	40 667	....	310	......	......	......	0.16	0.55	0.49	0.62	0.39	0.31	0.07	......	......	2.59	0.32	2.91	0.12	58	88	10	2	45	14	41
1921	40 542	....	316	......	......	0.05	0.35	0.38	0.40	0.63	0.45	0.20	0.07	......	......	2.53	0.23	2.76	0.06	60	88	7	5	44	17	39
1922	39 300	....	338	......	......	......	0.09	0.53	0.53	9.65	0.44	0.36	0.09	0.02	......	2.71	0.21	2.92	0.30	59	88	7	5	43	19	38
1923	39 120	....	338	......	......	0.02	0.19	0.57	0.42	0.69	0.50	0.31	0.10	0.03	......	2.83	0.29	3.12	0.20	58	87	9	4	42	22	36
1924	38 310	....	338	......	......	0.02	0.41	0.67	0.62	0.75	0.58	0.30	0.03	0.02	......	3.40	0.13	3.53	0.15	59	85	11	4	39	4	57
1925	36 696	65 381	333	......	......	0.01	0.37	0.68	0.57	0.74	0.54	0.31	0.05	0.05	......	3.32	0.50	3.82	0.16	59	82	14	4	43	6	51
1926	39 834	65 277	333	......	......	0.06	0.43	0.75	0.64	0.63	0.58	0.24	0.04	0.03	......	3.40	0.16	3.56	0.17	60	81	16	3	39	4	57

[1] Climatological records at Fallon, Nev.

[2] Based on crop report for entire project. Grain percentage would be slightly higher for Carson Division alone.

[3] Water shortage; Lahontan storage first available in 1914.

[4] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[5] Losses below Carson River diversion dam.

[6] Actual losses estimated as averaging 5% less than shown, due to under-measurement of farm deliveries.

[7] Actual deliveries estimated as averaging 5% more than shown, due to under-measurement.

[8] Final irrigable area not determined, but may not exceed 70 000 acres.

TABLE 12.—USE OF WATER ON THE NORTH PLATTE IRRIGATION PROJECT OF NEBRASKA AND WYOMING.

(Average elevation, 4 100 ft.; chiefly in sandy loam soil; practically no lining.)

Calendar year.	Irrigated area, total for year.[9]	Area commanded by constructed canals.[9]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[1]													Precipitation, May to September, in feet.[2]	Water delivered plus precipitation, May to September, in feet.	Precipitation, October to April preceding, in feet.[2]	Mean temperature, May to September, in degrees Fahrenheit.[2]	Percentage of Area in Different Crops.			Percentage of Total Diversions for Season.[5]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Canal and lateral losses.[6]	Waste.[7]	Delivered to farms.[8]
1912	50 250	70 000	602	......	......	......	......	0.15	0.75	0.72	0.23	0.40	......	......	......	2.25	1.23	3.48	0.53	61	47	33	20	39	1	60
1913	56 829	75 500	648	......	....	......	......	0.17	0.60	0.58	0.71	0.43	......	......	......	2.49	1.00	3.49	0.32	66	60	25	15	38	2	60
1914	60 532	75 500	651	......	......	......	......	0.31	0.56	0.78	0.80	0.45	0.02	......	......	2.92	0.38	3.30	0.56	66	62	17	21	45	1	54
1915	70 007	95 500	798	......	......	......	......	0.02	0.25	0.58	0.31	0.22	......	......	......	1.38	1.45	2.83	0.64	61	53	17	30	50	1	49
1916	75 620	96 000	800	......	......	......	......	0.29	0.40	0.56	0.60	0.32	......	......	......	2.17	0.65	2.82	0.24	64	51	24	25	43	1	56
1917	83 203	96 000	805	......	......	......	.....	0.01	0.32	0.64	0.69	0.47	......	......	......	2.13	0.87	3.00	0.46	63	49	22	29	42	1	57
1918[3]	93 636	107 500	882	......	......	......	......	0.15	0.50	0.67	0.63	0.29	......	......	......	2.24	1.21	3.45	0.46	66	47	30	23	46	1	53
1919	95 248	107 500	882	......	......	......	......	0.23	0.61	0.65	0.57	0.22	......	......	......	2.28	0.62	2.90	0.42	67	42	29	29	46	6	48
1920	96 535	112 500	935	......	......	......	......	0.00	0.39	0.66	0.58	0.36	......	......	......	1.99	0.68	2.67	0.76	65	38	24	38	43	8	49
1921	99 199	112 500	942	......	....	......	......	0.16	0.21	0.67	0.67	0.39	......	....	......	2.10	0.75	2.85	0.37	67	36	27	37	45	5	50
1922	110 305	146 300	1 144	......	......	......	....	0.10	0.52	0.72	0.69	0.49	......	......	......	2.52	0.62	3.14	0.52	67	32	27	41	42	8	50
1923[4]	128 795	172 200	1 339	......	......	......	......	0.02	0.19	0.57	0.47	0.45	......	......	.....	1.70	1.11	2.81	0.46	65	29	23	48	41	12	47
1924	130 121	206 200	1 451	......	......	......	......	0.19	0.52	0.80	0.90	0.44	......	......	......	2.85	0.53	3.38	0.51	64	29	23	48	41	15	44
1925	162 698	225 000	1 579	......	......	......	......	0.24	0.44	0.72	0.59	0.38	......	......	......	2.37	0.84	3.21	0.35	66	27	29	44	41	12	47
1926	170 831	237 000	1 579	......	......	......	......	0.10	0.36	0.59	0.70	0.37	......	......	......	2.12	0.90	3.02	0.42	65	30	29	41	.....	....	....

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[1] Does not include North Platte Canal and Colonization Company lands.

[2] Climatological records average of those at Mitchell (near) and Scottsbluff, Nebr.

[3] Fort Laramie unit included, 1918 and later years.

[4] Northport unit included, 1923 and later years.

[5] Total diversions from North Platte River at Whalen less waste at sand traps near diversion dam plus Northport Division diversions from Tri-State Canal plus outflow from Reservoirs 1, 2, and 3, minus inflow to these reservoirs. (Reservoirs 1, 2, and 3 under main canal of Interstate Diversion.)

[6] Does not include losses in Reservoirs 1, 2 and 3, losses above Northport diversion in Tri-State Canal, nor lateral losses in North Platte Canal and Colonization System (about 10% of total area), but does include losses on Lingle power diversions.

[7] Most of waste in upper reaches of main canals and utilized by other irrigators.

[8] Diversions to North Platte Canal and Colonization laterals considered as farm deliveries.

[9] Exclusive of North Platte Canal and Colonization Company lands, 16 000 acres irrigable, receiving water from Interstate Canal near its head; final irrigable area, 237 000 acres.

TABLE 13.—USE OF WATER ON THE OKANOGAN IRRIGATION PROJECT OF WASHINGTON.

(Average elevation, 1 000 ft.; 39 miles concrete-lined;* remainder in gravelly soil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.													Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March, preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canals and lateral losses.	Waste.	Delivered to farms.
1912	7 263	10 051	46	.....	......	......	......	0.10	0.43	0.43	0.28	......	......	......	......	1.24	0.74	1.98	0.72	60	15	....	21	64	47	1	52
1913	7 700	10 084	44	......	......	.....	......	0.20	0.30	0.49	0.56	0.01	......	......	......	1.56	0.53	2.09	0.31	62	18	....	8	74	38	0	62
1914	7 740	10 099	67†	......	......	......	......	0.55	0.62	0.64	0.74	0.04	......	......	......	2.59	0.41	3.00	0.66	62	21	....	3	76	28	1[4]	71
1915	7 800	10 099	72	......	......	......	0.02	0.30	0.55	0.61	0.76	0.10	0.02	0 02	......	2.38	0.50	2.88	0.62	62	22	....	8	70	24	1	75
1916	7 850	10 099	75	......	......	......	......	0.31	0.55	0.46	0.76	0.37	0.01	0.04	......	2.50	0.63	3.13	0.69	60	23	....	7	70	22[3]	2	76
1917	8 000	10 099	79	......	......	......	....	0.06	0.66	0.81	0.77	0.17	0.02	.....	......	2.49	0.29	2.78	0.34	62	25	1	8	66	20	1	79
1918	6 402	10 099	69	......	......	.....	......	0.23	0.32	0.16	0.10	0.15	0.03	.....	[2]	0.99	0.42	1.41	0.49	63	23	2	5	70	34	2	64
1919	5 849	10 099	65	......	......	.....	......	0.33	0.39	0.48	0.40	0.10	......	......	[2]	1.70	0.30	2.00	0.58	62	23	....	3	74	28	0	72
1920	5 436	8 599	66	......	......	......	......	0.05	0.21	0.33	0.23	0.11	0.04	......	[2]	0.97	0.44	1.41	0.24	62	15	....	3	82	38[5]	0	62
1921	5 644	8 599	67	......	......	......	......	0.39	0.68	0.73	0.74	0.40	......	0.01	......	2.95	0.29	3.24	0.76	62	14	....	3	83	24	0	76
1922	5 569	8 047	69	......	......	......	.....	0.35	0.69	0.70	0.77	0.24	......	......	[2]	2.75	0.42	3.17	0.46	64	11	....	4	85	28	0	72
1923	5 162	7 654	69	......	......	......	0.01	0.44	0.16	0.73	0.73	0.57	......	......	[2]	2.64	0.70	3.34	0.31	63	8	....	4	88	33	1	66
1924	4 940	7 589	70	.....	......	......	0.06	0.37	0.35	0.47	0.36	0.18	0.01	......	[2]	1.80	0.29	2.09	0.34	63	9	....	5	86	35[5]	0	65
1925	4 976	7 589	70	......	......	......	......	0.28	0.41	0.57	0.56	0.39	......	......	......	2.21	0.28	2.49	0.46	64	4	0	3	93	30	0	70
1926	4 581	7 300	70	......	......	......	0.06	0.33	0.42	0.52	0.43	0.25	......	......	[2]	2.01	0.37	2.38	0.49	64	2	....	3	95	24	0	76

* Six miles lined in 1912 and increased gradually to 39 miles in 1925.

† Distribution system extended to provide individual farm deliveries. Shrinkage in irrigable and irrigated areas resulted from inadequate water supply which became evident in the 1918–26 period of sub-normal stream flow.

[1] Climatological record at Omak, Wash.

[2] Average for all lands under project; shortages for the new lands occurred in 1918–20, 1922–24, and 1926.

[3] Includes some water used in October and November for concrete lining and unrecorded deliveries.

[4] Some project waste combined with reservoir spill in records.

[5] Percentage of losses increased by shortage of water.

TABLE 14.—USE OF WATER ON THE ORLAND IRRIGATION PROJECT OF CALIFORNIA.

(Average elevation, 250 ft.; 89 miles concrete-lined; remainder in clay and loam soil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[7]	Miles of canals and laterals operated.[3]	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.													Precipitation, March to October, inclusive, in feet.[1]	Water delivered plus precipitation, March to October, inclusive, in feet.	Precipitation, November to February preceding, in feet.[1]	Mean temperature, March to October, inclusive, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.	Waste.	Delivered to farms.
1912	4 230	14 200	81	......	......	0.05	0.28	0.68	0.81	0.78	0.76	0.29	0.30	......	......	3.95	0.93	4.88	0.30	67	82	....	8	10	20	31	49
1913	6 616	14 300	84	......	......	0.31	0.36	0.67	0.62	0.70	0.34	......	......	......	......	3.00	0.35	3.35	0.47	71	85	....	5	10	....	....	....
1914	7 354	14 300	86	......	......	0.03	0.22	0.73	0.74	0.80	0.78	0.60	0.18	......	......	4.08	0.26	4.34	2.13	69	82	....	6	12	....	....	....
1915	8 928	20 320	108	......	......	......	0.01	0.21	0.80	0.76	0.69	0.67	0.26	......	......	3.40	0.52	3.92	1.75	71	74	....	10	16	27	15	58
1916	9 357	20 533	121	......	......	......	0.53	0.79	0.80	0.72	0.70	0.51	0.02	......	......	4.07	0.35	4.42	1.27	69	65	....	16	19	20	16	64
1917	12 729	20 480	128	......	0.06	0.10	0.37	0.56	0.56	0.70	0.64	0.47	0.02	0.01	......	3.49	0.12	3.61	0.76	71	54	....	30	16	27	13	60
1918	14 764	20 480	128	......	......	......	0.18	0.48	0.37	0.33	0.24	0.23	0.08	......	[2]	1.91	0.61	2.52	0.60	69	50	9	25	16	33	5	62
1919	15 203	20 480	128	......	......	......	0.33	0.58	0.56	0.59	0.53	0.35	0.02	......	......	2.96	0.24	3.20	0.90	69	48	11	22	19	32	5	63
1920	13 872	20 533	128	0.01	0.06	0.12	0.35	0.34	0.26	0.20	0.11	0.04	......	......	[2]	1.49	0.44	1.93	0.28	68	56	9	13	22	37[4]	2	61
1921	14 697	20 533	139	......	......	......	0.34	0.50	0.51	0.57	0.56	0.43	0.09	0.01	......	3.01	0.37	3.38	1.41	68	53	7	17	23	26	9	65
1922	15 119	20 665	139	.....	......	......	0.47	0.54	0.57	0.64	0.59	0.49	0.04	......	......	3.34	0.26	3.60	0.99	69	56	5	14	25	25	9	66
1923	15 500	20 665	139	......	......	0.24	0.17	0.64	0.52	0.56	0.54	0.36	0.01	0.02	......	3.06	0.42	3.48	0.90	69	51	5	15	29	25	10	65
1924	11 962	20 659	114	......	0.02	0.10	0.25	0.26	0.12	0.01	0.01	0.01	0.01	0.06	0.03[2]	0.88	0.46	1.34	0.47	70	62	3	4	31	26[5]	13[6]	61
1925	13 955	20 659	137	0.09	0.05	0.20	0.08	0.39	0.60	0.77	0.65	0.43	0.09	0.04	......	3.39	0.71	4.10	1.22	69	50	4	16	30	27	11	62
1926	14 674	20 754	133	0.01	......	0.22	0.14	0.57	0.65	0.60	0.46	0.29	0.03	0.02	......	2.98	0.50	3.48	0.96	72	58	4	18	20	25	5	70

[1] Climatological records at Orland, Calif.

[2] Severe water shortages, supply in some other years slightly limited.

[3] Covers distribution system only, Seven Mile Reservoir feeder canal omitted.

[4] High rate of loss due to water shortage and resulting small canal discharges.

[5] Severe water shortage but rate of loss normal as most of canal system was dry from July 8 to November 9.

[6] Practically all waste occurred in November and December, after water shortage was over.

[7] Final irrigable area, 20 659 acres; irrigated area in past limited by water supply; additional storage to become available in 1929.

TABLE 15.—USE OF WATER ON THE RIO GRANDE IRRIGATION PROJECT OF NEW MEXICO AND TEXAS.

(Average elevation, 3 700 ft.; 10 miles concrete-lined, remainder chiefly in silt.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[11]	Miles of canals and laterals operated.[9]	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[2 4]													Precipitation, February to November, in feet	Water delivered plus precipitation, February to November, in feet.	Precipitation, December to January, preceding, in feet.[1]	Mean temperature, February to November, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.[5 8]	Waste.[6]	Delivered to farms.[10]
1912	23 115	25 000	6	......	......	....	......	.....	......	......	......	......	......	......	......	....	0.77	......	0.03	64	72	6	20	2	....	....	....
1913	27 723	35 000	37	......	0.19	0.83	0.86	1.07	0.93	0.22	0.05	0.04	0.16	......	......	4.35	0.67	5.02	0.08	64	71	7	18	4	7[5]	10	83
1914	28 442	40 000	37	0.17	0.39	0.52	0.82	0.96	0.80	0.47	0.80	0.47	0.28	......	......	5.68	0.88	6.56	0.08	65	64	8	22	5	26[7]	7	74
1915	33 876	45 000	37	......	0.05	0.15	0.91	0.99	1.18	1.02	0.77	0.56	0.17	......	0.10	5.90	0.64	6.54	0.39	64	66	9	20	5	5	17	78
1916	62 513	85 000	63	......	0.47	0.77	0.65	0.98	1.11	1.05	0.67	0.70	0.28	......	0.05	6.73	0.59	7.32	0.06	66	63	14	22	2	6	7	87
1917	64 308	88 000	86	0.01	0.37	0.72	0.85	0.93	1.18	1.13	0.77	1.05	0.68	0.29	0.40	8.38	0.48	8.86	0.04	65	42	22	34	2	....	....	....
1918	64 781	92 300	153	......	0.11	0.65	0.84	0.97	0.98	0.95	0.75	0.70	0.09	0.11	......	6.15	0.51	6.66	0.07	65	47	19	32	2	15[5]	20	65
1919	70 012	102 850	310	......	......	0.11	0.30	0.44	0.50	0.40	0.44	0.22	0.03	0.06	[3]	2.50	0.72	3.22	0.08	65	48	18	32	2	30	41	29
1920	82 956	115 000	397	0.03	0.03	0.22	0.45	0.57	0.43	0.44	0.48	0.33	0.13	0.03	0.05	3.09	0.53	3.62	0.09	65	43	14	41	2	30	40	30
1921	77 151	115 000	416	......	0.02	0.23	0.37	0.42	0.44	0.37	0.29	0.23	0.11	0.06	0.01	2.55	0.59	3.14	0.00	66	48	15	35	2	35	40	25
1922	74 325	116 000	416	......	0.06	0.13	0.31	0.30	0.43	0.45	0.45	0 35	0.15	0.04	0.04	2.71	0.36	3.07	0.05	66	47	11	40	2	41	38	21
1923	86 583	120 000	537	......	0.02	0.08	0.32	0.41	0.41	0.35	0.29	0.19	0.08	0.02	0.01	2.18	0.66	2.84	0.05	65	38	3	57	2	30	51	19
1924	110 318	142 000	584	......	0.02	0.09	0.34	0.38	0.38	0.23	0.38	0.23	0.07	0.02	0.02	2.18	0.48	2.66	0.10	66	31	1	66	2	38	44	18
1925	130 911	150 000	607	......	0.05	0.18	0.38	0.34	0.34	0.37	0.23	0.20	0.03	0.02	0.02	2.16	0.56	2.72	0.04	66	22	....	77	1	43	34	23
1926	142 523	150 000	612	......	0.07	0.13	0.27	0.26	0.33	0.24	0.35	0.26	0.01	0.01	0.01	1.91	0.93	2.84	0.07	66	21	1	77	1	47	28	25

[1] Mean of climatological records at El Paso, Tex., and Agricultural Coll., New Mexico.

[2] Includes losses in community ditches, 1912 to 1918, by considering deliveries to community ditches as deliveries to farms. No data on use under community canals.

[3] Charges for water deliveries changed from flat rate to acre-feet basis which was thereafter used.

[4] Project charges based on these deliveries; actual deliveries estimated as 25% greater.

[5] Loss in community ditches not included, 1912 to 1918.

[6] Returned to river and largely available for re-diversion.

[7] Waste unmeasured, included in loss.

[8] Actual losses estimated as averaging 7% less than shown, due to under-measurement of deliveries.

[9] Mileage increase disproportionate to irrigated area, due to gradual taking over of community ditches.

[10] Actual deliveries estimated as averaging 7% more than shown, due to under-measurement.

[11] Final irrigable area, 155 000 acres.

TABLE 16.—USE OF WATER ON THE FRANNIE DIVISION OF THE SHOSHONE IRRIGATION PROJECT, WYOMING.

(Average elevation 4 150 ft.; in sand and clay loam; no lining.)

Calender year.	Irrigated area, total for year.	Area commanded by constructed canals.[4]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[1]													Precipitation, April to September, in feet.[5]	Water delivered plus precipitation, April to September, in feet.	Precipitation, October to March, preceding, in feet.	Mean temperature, April to September, in degrees Fahrenheit.	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.[2]	Waste.	Delivered to farms.[3]
1917	2 300	5 000	.....	......	......	......	......	......	......	......	......	......	......	......	......	0.53	0.29	0.82	0.10	58	....	95	5	....	....	....	....
1918	4 729	13 007	......	......	......	......	......	0.08	0.86	1.00	0.72	0.19	......	......	......	2.85	0.50	3.35	0.08	58	6	91	3	....	....	....	....
1919	6 943	17 500	......	......	......	......	0.04	0.45	0.82	0.98	0.53	0.23	......	......	......	3.05	0.11	3.16	0.12	62	21	70	9	....	....	....	....
1920	10 460	22 484	......	......	......	......	......	0.16	0.66	0.82	0.66	0.29	0.04	......	......	2.63	0.16	2.79	0.22	59	42	47	9	....	....	....	....
1921	10 844	27 473	......	......	......	......	0.11	0.44	0.54	0.73	0.41	0.23	0.17	......	......	2.63	0.28	2.91	0.10	61	63	29	8	....	....	....	....
1922	10 152	27 410	175	......	......	......	......	0.21	0.73	0.71	0.53	0.29	0.14	......	......	2.61	0.36	2.97	0.08	61	72	20	8	....	33	17	50
1923	8 197	27 057	170	......	......	......	......	0.28	0.78	0.82	0.52	0.27	0.03	......	......	2.70	0.46	3.16	0.11	60	75	16	9	....	38	17	45
1924	6 750	20 000	171	......	......	......	......	0.34	0.41	0.47	0.47	0.19	0.01	......	......	1.89	0.17	2.06	0.14	58	72	15	13	....	46	24	30
1925	7 054	20 063	171	......	......	......	0.10	0.39	0.33	0.64	0.42	0.14	......	......	......	2.02	0.44	2.46	0.07	61	67	22	11	....	45	28	27
1926	7 663	20 063	143	......	......	......	0.01	0.21	0.53	0.52	0.39	0.06	......	......	......	1.72	0.52	2.24	0.13	59	71	14	15	....	49	17	34

[1] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[2] Actual losses estimated as averaging 5% less than shown, due to under-measurement of deliveries. Available data prior to 1922 do not segregate losses for Frannie Division. Indicated data after 1921 are based on losses within division only.

[3] Actual deliveries estimated as averaging 5% more than here shown, due to under-measurement.

[4] Final irrigable area, 28 000 acres; indicated reduction in past commanded areas due to elimination of poor land.

[5] Climatological records at Deaver, Wyo.

TABLE 17.—USE OF WATER ON THE GARLAND DIVISION OF THE SHOSHONE IRRIGATION PROJECT, WYOMING.

(Average elevation, 4 400 ft.; in gravel and loam; 4 miles of tunnel and other concrete-lined conduit.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[6]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[1]													Precipitation, April to September, in feet.[7]	Water delivered plus precipitation, April to September, in feet.	Precipitation, October to March preceding, in feet.	Mean temperature, April to September, in degrees Fahrenheit.	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Canal and lateral losses.[2,3]	Waste.	Delivered to farms.[5]
1912	16 524	41 331	242	......	......	......	0.00	0.22	0.58	0.48	0.27	0.10	0.01	......	......	1.66	0.49	2.15	0.07	56	60	37	3	...	36	8	56
1913	19 423	41 172	245	......	......	......	0.01	0.40	0.41	0.52	0.48	0.19	0.07	......	......	2.08	0.32	2.40	0.12	60	66	31	3	....	36	5	59
1914	22 226	41 166	245	......	......	......	0.04	0.35	0.45	0.72	0.48	0.15	0.19	......	......	2.38	0.46	2.84	0.06	59	67	30	3	....	37	5	58
1915	25 753	42 816	247	......	......	......	0.19	0.31	0.34	0.61	0.31	0.23	0.01	0.12	......	2.12	0.71	2.83	0.05	57	58	35	7	....	35	8	57
1916	29 977	42 623	266	......	......	......	0.07	0.32	0.46	0.75	0.45	0.23	0.01	0.05	......	2.34	0.25	2.59	0.07	58	54	40	6	....	35	9	56
1917	32 764	43 263	271	......	......	......	0.00	0.08	0.52	0.80	0.48	0.15	0.01	0.06	......	2.10	0.45	2.55	0.16	57	52	41	7	....	28	5	67
1918	33 552	42 299	280	......	......	......	......	0.27	0.68	0.55	0.44	0.17	......	.....	......	2.11	0.60	2.71	0.09	57	52	43	5	....	37	10	53
1919	34 697	43 400	280	......	......	.....	0.08	0.58	0.61	0.81	0.42	0.27	......	......	......	2.77	0.07	2.84	0.07	61	52	41	7	....	34	10	56
1920	35 175	43 284	280	......	......	......	......	0.29	0.69	0.66	0.53	0.32	......	......	......	2.49	0.14	2.63	0.07	57	61	30	9	....	36	6	58
1921	34 571	43 649	280	......	......	......	0.30	0.11	0.53	0.69	0.45	0.20	0.14	......	......	2.43	0.21	2.64	0.02	60	63	26	11	....	44	6	50
1922	32 717	43 813	282	......	......	......	......	0.23	0.59	0.67	0.41	0.21	0.13	......	......	2.24	0.46	2.70	0.08	60	58	26	16	....	45	5	50
1923	30 468	43 288	282	......	......	......	......	0.36	0.54	0.72	0.44	0.20	......	......	......	2.26	0.35	2.61	0.13	59	61	23	16	....	41	7	52
1924	29 757	42 000	279	......	......	......	......	0.49	0.52	0.63	0.68	0.24	0.01	......	......	2.57	0.18	2.75	0.14	57	66	16	18	....	41	5	54
1925	29 649	41 923	277	.....	......	......	0.14	0.50	0.51	0.80	0.54	0.26	0.01	......	......	2.76	0.34	3.10	0.08	60	66	18	16	....	39	7	54
1926	30 457	41 923	277	......	......	......	......	0.29	0.53	0.63	0.50	0.13	0.01	......	......	2.09	0.55	2.64	0.09	58	60	13	27	....	[4]	[4]	....

[1] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[2] Includes losses from Ralston Regulating Reservoir. From 1918-21, inclusive, represents combined loss on Garland and Frannie Divisions. After 1921, represents losses within Garland Division alone, with aggregate quantity of water delivered to Frannie Division deducted from diversion.

[3] Actual losses estimated as averaging 6% less than shown, due to under-measurement of deliveries.

[4] Losses and waste not segregated, the total amounting to 46 per cent.

[5] Actual deliveries estimated as averaging 6% more than shown, due to under-measurement.

[6] Final irrigable area, 42 000 acres; indicated past reductions in commanded areas, due to elimination of poor land.

[7] Climatological records at Powell, Wyo.

TABLE 18.—USE OF WATER ON THE FORT SHAW DIVISION OF THE SUN RIVER IRRIGATION PROJECT, MONTANA.

(Average elevation, 3 700 ft.; wholly in earth.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canal system.[6]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[2]													Precipitation, May to October, in feet.[1]	Water delivered plus precipitation, May to October, in feet.	Precipitation, November to April preceding, in feet.[1]	Mean temperature, May to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Canal and lateral losses.[3]	Waste.[4]	Delivered to farms.[5]
1912	6 824	16 346	120	......	......	......	......	......	0.80	0.72	0.18	0.01	......	......	......	1.71	0.73	2.44	0.14	......	23	73	4	....	20	27	53
1913	7 419	16 346	120	......	......	......	......	0.02	0.52	0.57	0.22	0.18	......	......	......	1.51	0.82	2.33	0.20	58	38	59	3	....	38	16	46
1914	6 613	16 346	110	......	......	......	......	0.03	0.29	0.92	0.36	0.12	0.01	......	......	1.73	0.71	2.44	0.22	59	51	46	3	....	40	14	46
1915	4 261	16 346	100	......	......	......	......	0.54	0.17	0.05	0.24	0.09	......	....	......	1.09	1.16	2.25	0.24	56	63	33	4	....	44	28	28
1916	4 717	16 332	100	......	......	......	......	0.14	0.26	0.14	0.58	0.09	0.01	......	....	1.22	1.22	2.44	0.15	56	71	26	3	....	42	26	32
1917	6 675	16 224	100	......	......	......	......	0.01	0.15	0.78	0.41	0.01	.....	......	......	1.36	0.61	1.97	0.21	58	73	24	3	....	38	27	35
1918	7 569	16 095	96	......	......	......	......	0.21	0.68	0.32	0.26	0.01	......	.....	......	1.48	0.42	1.90	0.15	59	63	35	2	....	40	23	37
1919	8 186	14 532	98	......	....	......	0.03	0.50	0.58	0.62	0.27	0.08	......	......	......	2.08	0.48	2.56	0.16	58	75	23	2	....	34	15	51
1920	8 048	13 935	100	......	......	......	......	......	0.54	0.29	0.40	0.01	......	......	......	1.25	0.50	1.75	0.36	58	75	22	3	....	35	24	41
1921	8 912	13 785	100	......	......	......	......	0.02	0.55	0.53	0.30	0.04	0.02	......	......	1.46	0.50	1.96	0.14	59	76	20	4	....	36	23	41
1922	8 115	13 701	100	......	......	......	......	......	0.41	0 53	0.40	0 06	0.02	......	......	1.42	0.61	2 03	0.40	59	82	12	6	....	29	31	40
1923	6 470	13 912	95	......	......	......	......	0.05	0.56	0.31	0.43	0.08	0.03	......	......	1.46	1.10	2.56	0.23	56	79	16	5	....	37	28	35
1924	7 888	13 902	101	......	......	......	.....	0.31	0.34	0.55	0.44	0.06	0.01	......	......	1.71	0.46	2.17	0.17	57	80	10	10	....	39	38	23
1925	7 456	13 902	101	......	......	......	......	0.30	0.23	0.66	0.36	0.05	......	......	.....	1.60	0.69	2.29	0 34	56	76	15	9	....	37	29	34
1926	7 183	13 902	101	......	......	......	......	0.17	0.45	0.62	0.29	0.04	......	......	......	1.57	0.63	2.20	0.11	59	71	21	8	....	35	18	47

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[1] Climatological records at Fort Shaw, Mont.

[2] Project charges based on these deliveries; actual deliveries estimated as 5% greater.

[3] Actual losses estimated as averaging 2% less than shown, due to under-measurement of deliveries.

[4] Waste largely rediverted by prior rights on Sun River below Fort Shaw Division.

[5] Actual deliveries estimated as averaging 2% more than shown, due to under-measurement.

[6] Final irrigable area about 14 000 acres.

TABLE 19.—USE OF WATER ON THE GREENFIELDS DIVISION OF THE SUN RIVER IRRIGATION PROJECT, MONTANA.

(Average elevation, 3 700 ft.; lined in a few isolated stretches, remainder located in gravel and clay soil.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canal system.[7]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[1] [2]													Precipitation, May to October, in feet.	Water delivered plus precipitation, May to October, in feet.	Precipitation, November to April, preceding, in feet.	Mean temperature, May to October, in degrees Fahrenheit.[8]	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.[3]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Canal and lateral losses.[4] [5]	Waste.	Delivered to farms.[6]
1919	3 311	24 300	148	......	......	......	......	0.00	0.72	0.72	0.00	0.00	0.00	......	......	1.44	0.48	1.92	0.16	58	48	50	2	...	46	1	53
1920	6 729	24 300	150	......	......	......	......	0.00	0.33	0.87	0.14	0.02	0.00	......	......	1.36	0.45	1.81	0.36	58	14	85	1	...	34	21	45
1921	11 849	25 150	167	....	......	.	......	0.01	0.73	0.53	0.05	0.01	0.01	......	......	1.34	0.49	1.83	0.17	57	13	86	1	...	27	13	60
1922	12 422	27 473	167	......	......	......	......	0.00	0.21	0.82	0.06	0.01	0.02	......	......	1.12	0.69	1.81	0.50	59	13	86	1	...	11	26	63
1923	2 624	28 304	164	......	......	......	......	0.00	0.00	0.58	0.60	0.06	0.40	......	......	1.64	0.99	2.63	0.36	57	32	66	2	...	27	50	23
1924	13 741	42 920	236	......	......	......	......	0.21	0.29	0.69	0.14	0.01	0.02	......	......	1.34	0.51	1.85	0.27	56	20	77	3	...	32	23	45
1925	13 012	42 920	241	......	......	......	......	0.23	0.18	0.74	0.05	......	......	......	......	1.20	0.78	1.98	0.33	55	21	78	1	...	36	20	44
1926	15 958	41 975	251	......	......	......	......	0.16	0.38	0.28	0.01	0.00	......	......	......	0.83	0.78	1.61	0.12	56	25	73	2	...	34	21	45

[1] Data cover Part No. 1 of Greenfields Division only; in general, about 94% of indicated total irrigated area.

[2] Project charges based on these deliveries; actual deliveries estimated as 5% greater.

[3] Measured at Fairfield Drop, losses from Sun River Diversion Dam to Drop excessive on account of small part of division area irrigated.

[4] Below Fairfield Drop, only.

[5] Actual losses estimated as averaging 2% less than shown, due to under-measurement of deliveries.

[6] Actual deliveries estimated as averaging 2% more than shown, due to under-measurement.

[7] Final irrigable area, 100 000 acres.

[8] Climatological records at Fort Shaw, Mont. for 1919; other years at Fairfield, Mont.

TABLE 20.—USE OF WATER ON THE UMATILLA IRRIGATION PROJECT OF OREGON.

(Average elevation, 470 ft.; 157 miles of pipe or lined canal, remainder in sandy loam.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[9]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[2]													Precipitation, March to October, in feet.[1]	Water delivered plus precipitation, March to October, in feet.	Precipitation, November to February, preceding, in feet.[1]	Mean temperature, March to October, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.[4]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.[3] [7]	Waste.	Delivered to farms.[8]
1912	4 600	17 253	100	......	......	......	0.96	1.52	1.65	1.78	1.62	0.72	......	......	......	8.25	0.43	8.68	0.31	59	64	....	13	23	28	2[6]	70
1913	5 006	16 652	112	......	......	0.02	1.09	1.59	1.71	1.31	1.63	1.10	......	......	......	8.45	0.38	8.83	0.23	60	47	14	8	31	27	2	71
1914	5 100	17 000	120	......	......	0.21	1.09	1.38	1.42	1.29	1.20	0.52	......	......	......	7.11	0.27	7.38	0.39	62	60	4	8	28	40	2	58
1915	5 306	17 000	127	......	......	0.05	0.73	0.66	1.26	1.17	1.15	0.53	0.02	......	......	5.57	0.30	5.87	0.19	63	66	2	7	25	44	28	28
1916	5 477	17 000	139	......	......	......	0.44	0.99	1.15	1.05	1.26	0.82	0.05	.....	......	5.76	0.53	6.29	0.58	59	75	....	7	18	36	8	56
1917	7 327	26 300	164	......	......	......	0.25	0.88	1.34	1.50	1.27	0.64	0.26	0.05	......	6.19	0.31	6.50	0.38	59	78	....	7	15	25[5]	23	52
1918	9 100	24 732	167	......	......	......	0.77	1.02	0.98	0.94	0.82	0.65	0.11	......	......	5.29	0.28	5.57	0.40	61	82	1	6	11	32	16	52
1919	10 533	24 625	167	......	......	0.05	0.62	1.08	0.99	0.98	0.95	0.44	0.13	......	......	5.24	0.24	5.48	0.36	60	86	1	5	8	33	23	44
1920	12 028	24 464	168	......	......	......	0.18	1.05	0.77	1.09	0.87	0.21	0.04	......	......	4.21	0.53	4.74	0.39	60	89	....	4	7	31	25	44
1921	13 145	24 400	174	......	......	......	0.60	0.89	0.86	0.91	0.84	0.26	0.01	......	......	4.37	0.25	4.62	0.48	60	89	1	4	6	31	24	45
1922	13 273	24 592	180	......	......	......	0.21	0.92	0.93	1.09	0.83	0.47	0.02	......	......	4.47	0.30	4.77	0.44	61	88	1	6	5	34	18	48
1923	13 330	24 587	188	......	......	......	0.51	1.02	0.78	0.94	0.95	0.44	......	......	......	4.64	0.54	5.18	0.33	61	89	1	6	4	32	19	49
1924	13 134	24 587	193	......	......	......	0.75	0.92	0.69	1.11	0.91	0.14	......	......	......	4.52	0.28	4.80	0.25	61	89	1	6	4	36	12	52
1925	13 345	24 587	192	......	......	0.11	0.84	1.07	1.10	1.13	0.93	0.35	......	......	......	5.53	0.23	5.76	0.36	62	90	2	5	3	28	12	60
1926	12 549	24 587	192	......	......	0.12	1.02	0.98	1.08	1.00	0.77	0.29	......	......	......	5.26	0.33	5.59	0.35	62	91	3	5	1	28	8	64
1927	11 462	24 587	......	......	......	......	0.50	1.05	1.16	1.38	1.22	0.29	......	......	......	5.70	0.50	6.20	0.53	60	85	5	6	4	24	16	60

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[1] Climatological record at Hermiston, Ore.

[2] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[3] Not including feed canal, which is 25 miles long with an average annual loss of 9 per cent.

[4] Sum of discharges at heads of Maxwell, A, and West Division Main Canals.

[5] Loss in West Division Canals estimated at 2%; system is largely concrete-lined or piped.

[6] Includes some water used for construction purposes.

[7] Actual losses estimated as averaging 5% less than shown, due to under-measurement of deliveries.

[8] Actual deliveries estimated as averaging 5% more than shown, due to under-measurement.

[9] Final irrigable area, 28 300 acres.

TABLE 21.—USE OF WATER ON THE UNCOMPAHGRE IRRIGATION PROJECT OF COLORADO.

(Average elevation, 5 500 ft.; 11 miles concrete-lined tunnel and canal; about one-half of canals in adobe and shale, remainder in sand and gravel.)

Calendar year.	Irrigated area, total for year.[3]	Area commanded by constructed canals.[11]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[4] [5]													Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.[6]		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.[5] [7] [8]	Waste.	Delivered to farms.[10]
1912	27 887	44 500	211	......	......	......	[2]	0.93	1.02	0.83	0.92	0.72	0.39	......	......	4.81	0.64	5.45	0.33	56	43	22	23	12	5[9]	....	95
1913	31 428	48 000	228	......	......	......	[2]	0.89	1.04	0.98	0.98	0.75	0.45	......	......	5.09	0.33	5.42	0.16	61	41	24	26	9	5[9]	....	95
1914	33 873	52 338	280	......	......	......	[2]	0.81	1.07	1.05	0.91	0.71	0.51	......	......	5.06	0.85	5.91	0.38	60	39	33	20	8	8[9]	....	92
1915	41 463	62 147	356	.....	......	......	0.35	0.97	1.13	1.11	1.04	0.58	0.41	......	......	5.58	0.47	6.05	0.19	60	44	31	18	7	12[9]	....	88
1916	49 275	77 713	406	......	......	......	0.43	1.17	1.27	1.26	0.85	0.77	0.33	......	......	6.08	0.81	6.89	0.32	60	44	32	17	7	2	7	91
1917	53 108	90 000	415	......	......	......	0.23	0.72	1.34	1.35	1.11	0.77	0.44	......	......	5.96	0.53	6.49	0.24	59	42	31	22	5	3	11	86
1918	58 270	100 000	413	......	.....	......	0.39	1.32	1.31	1.20	0.95	0.68	0.45	......	......	6.30	0.55	6.85	0.22	60	43	32	21	4	7	6	87
1919	60 906	100 000	417	......	......	......	0.41	1.30	1.21	1.29	1.01	0.71	0.49	......	......	6.42	0.46	6.88	0.34	61	48	29	19	4	6	9	85
1920	64 186	100 000	448	......	......	......	0.14	0.88	1.27	1.17	1.06	0.86	0.32	......	......	5.70	0.68	6.38	0.26	59	52	24	21	3	5	12	83
1921	63 759	97 410	455	......	......	......	0.52	1.32	1.10	1.34	0.95	0.76	0.53	......	......	6.52	0.65	7.17	0.22	62	49	26	22	3	9	12	79
1922	64 730	97 410	468	......	......	......	0.45	1.21	1.33	1.12	1.08	0.84	0.50	......	......	6.53	0.29	6.82	0.26	61	47	21	29	3	5	9	86
1923	64 324	97 064	488	......	......	......	0.45	0.93	1.16	1.14	0.74	0.56	0.13	......	......	5.11	0.55	5.66	0.25	60	45	24	28	3	21	12	67
1924	62 184	95 202	532	......	......	......	0.39	1.11	1.00	1.10	0.76	0.45	0.08	......	......	4.89	0.52	5.41	0.26	60	46	18	33	3	20	9	71
1925	61 637	95 202	532	......	......	......	0.58	1.16	0.98	1.11	0.72	0.18	0.11	......	......	4.84	0.64	5.48	0.26	62	48	22	27	3	27	11	62
1926	58 676	95 202	532	......	......	......	0.36	1.20	1.12	1.23	0.84	0.48	0.09	......	......	5.32	0.62	5.94	0.28	62	46	24	28	2	23	10	67

[1] Climatological records at Montrose, Colo.

[2] Not recorded.

[3] Additional area irrigated from private canals contracted to be turned over to the U. S. Bureau of Reclamation and supplied with Gunnison water: 1912, 4513 A; 1913, 6054 A; 1914, 6000 A; 1915, 4500 A; 1916, 700 A; 1917-23, 1000 A.

[4] Project charges based on these deliveries; actual deliveries estimated 5% greater.

[5] Prior to 1923 water delivered on continuous flow basis. For 1923 and subsequent years water delivered on demand basis.

[6] Total diversions from Uncompahgre and Gunnison Rivers less Gunnison water delivered to Uncompahgre River.

[7] Losses small because of unmeasured seepage and farm waste entering canals and laterals. Some return flow charged to canals.

[8] Actual losses estimated to average 3% less than shown, due to under-measurement of deliveries.

[9] Waste included.

[10] Actual deliveries estimated to average 3% more than shown, due to under-measurement.

[11] Final irrigable area, 97 064 acres.

TABLE 22.—USE OF WATER ON THE SUNNYSIDE DIVISION OF THE YAKIMA IRRIGATION PROJECT, WASHINGTON.

(Average elevation, 800 ft.; in sand and volcanic ash; lined sections totaling 108 miles in 1915 and 158 miles in 1924.)

Calendar year.	Irrigated area, total for year.[2]	Area commanded by constructed canals.[10]	Miles of canals and laterals operated.	Delivered to Farms, in Hundredths of Acre-Feet per Acre.[4] [5]													Precipitation, April to October, in feet.[1]	Water delivered plus precipitation, April to October, in feet.	Precipitation, November to March, preceding, in feet.[1]	Mean temperature, April to October, in degrees Fahrenheit.[1]	Percentage of Area in Different Crops.				Percentage of Total Diversions for Season.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.[7] [8]	Waste.	Delivered to farms.[9]
1912	62 800	80 076	525	......	......	......	0.26	0.52	0.55	0.59	0.55	0.34	0.26	......	......	3.07	0.42	3.49	0.23	60	51	1	20	28	....	....	....
1913	64 400	80 658	525	......	......	......	0.29	0.54	0.49	0.58	0.60	0.43	0.25	......	......	3.18	0.28	3.46	0.20	61	55	....	19	26	....	....	....
1914	66 525	81 807	525	......	......	......	0.29	0.55	0.53	0.58	0.57	0.37	0.18	......	......	3.07	0.41	3.48	0.38	61	54	1	19	26	....	....	....
1915	68 840	89 253	572	......	......	......	0.29	0.48	0.54	0.54	0.26	0.11	0.23	......	[3]	2.45	0.25	2.70	0.29	63	51	4	24	21	31	7	62
1916	73 000	93 226	585	......	......	......	0.29	0.56	0.56	0.58	0.63	0.43	0.24	......	......	3.29	0.20	3.49	0.52	61	55	3	21	21	26	5	69
1917	80 500	97 285	590	......	......	......	0.16	0.52	0.52	0.65	0.63	0.41	0.27	......	......	3.16	0.17	3.33	0.21	62	51	6	26	17	27	7	66
1918	84 650	98 537	590	......	......	......	0.34	0.60	0.59	0.63	0.61	0.44	0.22	......	......	3.43	0.24	3.67	0.35	64	57	4	24	15	23	7	70
1919	90 000	100 130	605	......	......	......	0.33	0.59	0.57	0.61	0.60	0.36	0.22	......	......	3.28	0.14	3.42	0.23	61	62	5	18	15	24	6	69
1920	93 610	100 733	605	......	......	......	0.26	0.58	0.53	0.51	0.59	0.37	0.20	......	......	3.04	0.35	3.39	0.23	61	65	7	14	14	24	7	69
1921	94 500	101 509	605	......	......	......	0.33	0.59	0.55	0.60	0.59	0.41	0.21	......	......	3.28	0.19	3.47	0.32	62	60	7	18	15	22	7	71
1922	95 000	101 339	605	......	......	......	0.15	0.58	0.59	0.63	0.60	0.42	0.21	......	......	3.18	0.17	3.35	0.36	63	55	6	23	16	23	6	71
1923	95 000	101 329	605	......	......	......	0.36	0.61	0.56	0.61	0.60	0.44	0.12	......	......	3.30	0.40	3.70	0.19	63	57	7	20	16	21	7	72
1924	95 000	102 346	605	......	......	0.02	0.46	0.59	0.62	0.63	0.61	0.34	0.11	......	......	3.38	0.07	3.45	0.21	65	58	6	19	17	22	6	72
1925	95 000	102 335	605	......	......	0.02	0.44	0.62	0.62	0.65	0.63	0.42	0.12	......	......	3.52	0.28	3.80	0.36	63	54	8	22	16	21	6	73
1926	94 000	102 464	605	......	......	0.08	0.52	0.63	0.57	0.63	0.55	0.24	0.07	......	......	3.29	0.15	3.44	0.28	63	54	9	23	14	21	7	72

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[1] Climatological records at Sunnyside, Wash.

[2] Irrigation districts included, 1916 and subsequent years.

[3] Shortage of storage water.

[4] Deliveries are not generally measured at farms. Lateral losses have been determined to average 15% and the indicated deliveries are 85% of quantities flowing at heads of laterals where water is always measured.

[5] Project charges based on these deliveries; actual deliveries estimated as 5% greater.

[6] Slight shortage.

[7] Lateral losses assumed 15% of discharge at heads of laterals; canals receive unmeasured inflow of waste and seepage water, which has gradually increased in recent years.

[8] Actual losses estimated to average 4% less than shown, due to under-measurement of deliveries.

[9] Actual deliveries estimated to average 4% more than shown, due to under-measurement.

[10] Final irrigable area, 107 600 acres.

TABLE 23.—USE OF WATER ON THE TIETON DIVISION OF THE YAKIMA IRRIGATION PROJECT, WASHINGTON.

(Average elevation, 1 500 ft.; in sandy loam and gravel; lined sections totaling 80 miles in 1913 and 92 miles in 1924.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[6]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[3]													Precipitation, April to September, in feet.[1]	Water delivered plus precipitation, April to September, in feet.	Precipitation, October to March preceding, in feet.[1]	Mean temperature, April to September, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.[4]	Waste.	Delivered to farms.[5]
1912	15 008	....	260	......	......	......	......	0.41	0.56	0.51	0.57	0.24	......	......	......	2.29	0.28	2.57	0.50	60	29	6	35	30	....	....	....
1913	18 750	....	335	.....	......	......	......	0.45	0.44	0.57	0.55	0.26	......	......	......	2.27	0.25	2.52	0.38	62	33	9	25	33	....	....	....
1914	20 600	30 000	335	......	......	......	......	0.46	0.34	0.54	0.53	0.22	.....	......	......	2.09	0.36	2.45	0.54	62	36	12	30	22	27	9	64
1915	22 000	30 000	335	......	......	......	......	0.31	0.47	0.51	0.43	0.12	......	......	[2]	1.84	0.28	2.12	0.57	63	41	18	19	22	27	8	65
1916	23 000	30 000	335	......	.....	......	......	0.43	0.46	0.42	0.52	0.32	......	......	......	2.15	0.21	2.36	0.76	60	46	20	13	21	27	7	66
1917	25 400	31 000	335	......	......	......	......	0.43	0.48	0.50	0.50	0.34	......	......	......	2.25	0.13	2.38	0.31	61	46	18	14	22	26	3	71
1918	26 400	31 000	335	......	......	.....	0.04	0.47	0.50	0.53	0.51	0.38	......	......	......	2.43	0.12	2.55	0.52	63	45	19	11	25	28	1	71
1919	27 000	32 000	335	......	......	......	0.14	0.54	0.50	0.54	0.53	0.37	......	......	......	2.62	0.10	2.72	0.43	62	52	12	10	26	27	1	72
1920	28 000	32 000	335	......	......	......	0.09	0.52	0.50	0.51	0.50	0.36	......	......	......	2.48	0.21	2.69	0.27	60	52	15	7	26	27	1	72
1921	28 500	32 000	335	......	......	......	0.05	0.49	0.49	0.54	0.54	0.37	0.02	......	......	2.50	0.15	2.65	0.64	60	50	12	7	31	25	4	71
1922	28 700	32 000	335	.... .	......	......	......	0.46	0.50	0.55	0.55	0.41	0.01	......	......	2.48	0.05	2.53	0.53	63	45	10	9	36	21	3	76
1923	28 350	32 000	335	......	......	......	0.06	0.55	0.48	0.49	0.55	0.41	0.01	......	......	2 55	0.40	2.95	0.47	62	44	9	10	37	21	4	75
1924	27 970	32 000	335	......	......	......	0.13	0.58	0.55	0.56	0.55	0.27	.....	......	......	2.64	0.10	2.74	0.36	63	43	6	11	40	21	1	78
1925	27 650	32 000	335	......	......	......	0.10	0.52	0.51	0 55	0.54	0.32	......	......	......	2.54	0.28	2.82	0.49	63	32	11	14	43	25	3	72
1926	28 100	32 000	335	......	......	......	0.29	0.53	0.52	0.54	0.54	0.22	......	......	......	2.64	0.06	2.70	0.41	62	34	9	14	43	22	1	77

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[1] Climatological records at Cowiche, Wash.

[2] Water shortage.

[3] Project charges based on these deliveries; actual deliveries estimated to average 5% greater.

[4] Actual losses estimated to average 4% less than shown, due to under-measurement of deliveries.

[5] Actual deliveries estimated to average 4% more than shown, due to under-measurement.

[6] Final irrigable area, 32 000 acres.

TABLE 24.—USE OF WATER ON THE YUMA IRRIGATION PROJECT OF ARIZONA AND CALIFORNIA.

(Average elevation, 120 ft.; in fine sandy loam.)

Calendar year.	Irrigated area, total for year.	Area commanded by constructed canals.[6]	Miles of canals and laterals operated.	DELIVERED TO FARMS, IN HUNDREDTHS OF ACRE-FEET PER ACRE.[2]													Precipitation, January to December, in feet.[1]	Water delivered plus precipitation, January to December, in feet.	Total annual precipitation, in feet.[1]	Mean temperature, January to December, in degrees Fahrenheit.[1]	PERCENTAGE OF AREA IN DIFFERENT CROPS.				PERCENTAGE OF TOTAL DIVERSIONS FOR SEASON.		
				January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Total.					Alfalfa, hay, and pasture.	Small grains.	Furrow crops.	Trees.	Canal and lateral losses.[3 4]	Waste.[3]	Delivered to farms.[5]
1912	13 767	....	171	0.19	0.41	0.36	0.39	0.44	0.41	0.50	0.52	0.51	0.34	0.22	0.21	4.60	0.26	4.86	0.26	71	69	14	17	....	....	....	....
1913	19 607	....	227	0.07	0.31	0.38	0.42	0.43	0.53	0.61	0.51	0.46	0.29	0.16	0.17	4.36	0.09	4.45	0.09	71	74	9	17	....	12	0	25
1914	25 207	....	250	0.20	0.14	0.34	0.33	0.37	0.44	0.44	0.45	0.49	0.26	0.10	0.07	3.63	0.31	3.94	0.31	73	70	9	21	....	....	....	....
1915	27 857	....	307	0.10	0.03	0.40	0.33	0.27	0.44	0.41	0.42	0.40	0.26	0.21	0.14	3.41	0.36	3.77	0.36	71	64	10	26	....	12	25	22
1916	29 483	....	316	0.07	0.12	0 28	0.31	0.33	0.45	0.39	0.48	0.36	0.22	0.12	0.07	3.20	0.20	3.40	0.20	72	59	6	35	....	19	22	24
1917	36 956	....	335	0.06	0.20	0.37	0.36	0.42	0.44	0.46	0.48	0.43	0.28	0.09	0.11	3.70	0.19	3.89	0.19	72	42	2	56	....	19	26	30
1918	45 670	....	338	0.05	0.06	0.31	0.33	0.30	0.40	0.57	0.56	0.42	0.22	0.04	0.03	3.29	0.24	3.53	0.24	72	26	2	72	....	16	20	32
1919	53 284	....	338	0.06	0.11	0.26	0.35	0.25	0.39	0.53	0.44	0.30	0.14	0.04	0.05	2.92	0.17	3.09	0.17	72	30	2	68	....	27	40	33
1920	54 550	....	340	0.04	0.02	0.45	0.28	0.20	0.35	0.59	0.51	0.22	0.16	0.06	0.06	2.94	0.40	3.34	0.40	71	28	1	71	....	10	25	34
1921	52 390	....	345	0.08	0.17	0.31	0.40	0.22	0.29	0.42	0.26	0.24	0.11	0.11	0.08	2.69	0.58	3.27	0.58	72	48	5	47	....	11	23	29
1922	53 970	....	345	0.03	0.14	0.34	0.20	0.23	0.31	0.33	0.35	0.22	0.24	0.12	0.08	2.59	0.25	2.84	0.25	72	54	2	44	....	10	24	26
1923	53 270	....	345	0.11	0.19	0.33	0.31	0.23	0.34	0.34	0.41	0.24	0.24	0.10	0.06	2.90	0.33	3 23	0.33	71	50	4	46	....	16	61	23
1924	53 207	....	324	0.06	0.19	0.36	0.34	0.22	0.44	0.53	0.54	0.33	0.22	0.08	0.08	3.39	0.07	3.46	0.07	72	45	1	54	....	14	59	27
1925	55 904	64 865	324	0.08	0.18	0.39	0.25	0.24	0.45	0.50	0.44	0.29	0.10	0.05	0.08	3.05	0.32	3.37	0.32	72	35	1	64	....	16	57	27
1926	60 290	64 865	324	0.07	0.19	0.38	0.21	0.20	0.41	0.42	0.28	0.15	0.15	0.11	0.02	2.59	0.77	3.36	0.77	73	36	5	59	....	14	63	23

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[1] Climatological records at Yuma, Ariz., Weather Bureau Station.

[2] Project charges based on these deliveries; actual deliveries estimated as 10% greater.

[3] Losses and waste from lateral system only, for 1912 to 1918, and from 1920 to 1922, loss and waste from main canal being inseparable. Percentage based on diversions from Colorado River at Laguna. Available water supply at all times exceeds project requirement, and it is customary to operate with canals largely filled to minimize silt deposition.

[4] Actual losses estimated as averaging 3% less than shown, due to under-measurement of deliveries.

[5] Percentage expressed in terms of diversions from Colorado River, at Laguna. Actual deliveries estimated as averaging 3% more than shown, due to under-measurement.

[6] Final irrigable area, 108 000 acres.

TABLE 25.—AVERAGE USE OF WATER ON FEDERAL IRRIGATION PROJECTS.

Detailed data in Table:	Project.	Irrigated area, in acres.	Area commanded by constructed canal system, in acres.	Miles of canals and laterals operated.	Miles of canals and laterals lined or enclosed.	Predominating character of soils.	Delivered (Charged) to Farms, in Hundredths of Acre-Feet per Acre.[10]						
							January.	February.	March.	April.	May.	June.	July.
1	Belle Fourche	45 164	74 569	547	58	Heavy	....	....	....	....	0.07	0.25	0.40
2	Boise[1]	145 616	163 667	1 004	37	Light	....	....	....	0.16	0.73	0.78	0.89
3	Carlsbad	22 535	25 000	45	11	Medium	0.04	0.06	0.09	0.51	0.29	0.28	0.37
4	Grand Valley[2]	10 139	29 000	180	7	Heavy	....	....	....	0.14	0.69	0.78	0.81
5	Huntley	19 406	32 540	232	....	Heavy	....	....	....	....	0.12	0.28	0.51
6	King Hill[3]	6 460	16 890	96	43	Very light	....	....	....	0.31	1.23	1.30	1.51
7	Klamath	43 325	52 396	240	2	Medium	....	....	....	0.01	0.31	0.41	0.35
8	Lower Yellowstone	17 540	48 272	202	....	Heavy	....	....	....	....	0.07	0.30	0.51
9	Milk River	16 793	63 480	275	....	Heavy	....	....	....	0.01	0.11	0.22	0.21
10	Minidoka-South Side Pumping Division	44 945	48 880	275	....	Medium	....	....	....	0.06	0.41	0.56	0.66
11	Newlands	38 808	65 277	319	....	Medium	....	....	0.03	0.28	0.58	0.53	0.64
12	North Platte	107 694	161 870	1 154	....	Medium	....	....	....	....	0.12	0.41	0.67
13	Okanogan[4]	5 260	7 300	68	39	Light	....	....	....	....	0.37	0.42	0.68
14	Orland[5]	14 554	20 600	135	89	Light	0.01	0.02	0.11	0.27	0.54	0.57	0.63
15	Rio Grande[6]	96 847	126 300	485	10	Medium	....	0.04	0.18	0.41	0.47	0.49	0.43
	Shoshone:												
16	Frannie Division[7]	7 963	20 063	166	....	Heavy	....	....	....	0.02	0.29	0.56	0.63
17	Garland Division[8]	32 380	42 000	279	4	Medium	....	....	....	0.05	0.30	0.57	0.71
	Sun River:												
18	Fort Shaw Division	7 650	13 902	99	....	Heavy	....	....	....	....	0.16	0.45	0.52
19	Greenfields Division[9]	9 867	41 975	190	....	Medium	....	....	....	....	0.08	0.35	0.65
20	Umatilla[2]	10 970	24 587	173	157	Light	....	....	0.02	0.52	0.98	0.96	1.07
21	Uncompahgre	61 178	95 202	470	11	Medium	....	....	....	0.39	1.11	1.18	1.21
	Yakima:												
22	Sunnyside Division	91 726	102 464	602	125	Medium	....	....	0.01	0.33	0.59	0.57	0.62
23	Tieton Division	27 607	32 000	335	86	Light	....	....	....	0.09	0.51	0.50	0.53
24	Yuma	51 950	64 865	336	....	Medium	0.06	0.15	0.35	0.30	0.25	0.38	0.47

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[1] 1924 and 1926 omitted from average on account of water shortages.
[2] Umatilla and Grand Valley data cover years, 1916-25.
[3] King Hill data cover years, 1921-27.
[4] Okanogan data average for 1921, 1923, and 1925, on account of shortages in all other years since 1917.
[5] Orland data omit years of heavy water shortage, 1918, 1920, and 1924.
[6] Rio Grande data cover years, 1919-26.
[7] Shoshone-Frannie Division data cover years, 1922-26.
[8] Losses on water conveyed through Garland Division Main Canals for Frannie Division included in Garland Division losses.
[9] 1919 to 1926, inclusive.
[10] Data are for years, 1917 to 1926, inclusive, except as noted.

TABLE 25.—(*Continued*).

Detailed data in Table:	Project.	Delivered (Charged) to Farms, in Hundredths of Acre-Feet per Acre.[10]						Precipitation, in growing season, in feet.	Water delivered plus precipitation in growing season, in feet.	Precipitation, preceding growing season, in feet.	Elevation of project, in feet.	Mean temperature in growing season, in degrees Fahrenheit.	Percentage of Area in Different Crops.				Percentage of Total Diversions.		
		August.	September.	October.	November.	December.	Total.						Alfalfa, hay, and pasture.	Small grain.	Furrow crops.	Trees.	Canal and lateral losses.	Waste.	Delivered to farms.
1	Belle Fourche	0.35	0.15	....	....	....	1.22	0.86	2.08	0.48	2 800	65	62	22	16	....	33	15	52
2	Boise[1]	0.71	0.29	0.04	....	....	3.60	0.38	3.98	0.43	2 500	62	53	31	13	3	28	2	70
3	Carlsbad	0.44	0.20	0.06	0.02	....	2.36	0.86	3.22	0.06	3 100	66	33	4	63	....	48	6	46
4	Grand Valley[2]	0.61	0.35	0.19	0.04	....	3.61	0.48	4.09	0.27	4 700	65	36	24	33	7	43	22	35
5	Huntley	0.37	0.11	....	....	....	1.39	0.63	2.02	0.37	3 000	64	42	34	24	....	36	30	34
6	King Hill[3]	1.48	0.97	0.20	0.01	....	7.01	0.35	7.36	0.41	2 750	64	75	7	11	6	......	....	53
7	Klamath	0.30	0.05	....	....	....	1.43	0.18	1.61	0.68	4 100	60	77	21	2	....	39	9	52
8	Lower Yellowstone	0.35	0.11	....	....	....	1.34	0.71	2.05	0.33	1 900	64	45	33	22	....	44	21	35
9	Milk River	0.07	0.02	0.01	....	....	0.65	0.91	1.56	0.18	2 200	58	73	22	5	....	36	19	45
10	Minidoka-South Side Pumping Division	0.56	0.27	0.02	....	....	2.54	0.53	3.07	0.32	4 200	59	50	28	22	....	39	3	58
11	Newlands	0.46	0.28	0.06	0.02	....	2.88	0.25	3.13	0.15	4 000	59	85	11	4	....	41	14	45
12	North Platte	0.65	0.38	....	....	....	2.23	0.81	3.04	0.47	4 100	66	36	26	38	....	43	8	49
13	Okanogan[4]	0.68	0.45	....	....	....	2.60	0.42	3.02	0.51	1 000	63	9	....	3	88	29	....	71
14	Orland[5]	0.57	0.40	0.04	0.01	....	3.17	0.37	3.55	1.02	250	70	53	5	19	23	27	9	64
15	Rio Grande[6]	0.42	0.30	0.10	0.03	0.02	2.89	0.60	3.49	0.06	3 700	66	37	8	53	2	32	39	29
	Shoshone:																		
16	Frannie Division[7]	0.46	0.19	0.04	....	....	2.19	0.39	2.58	0.11	4 150	60	72	17	11	....	42	21	37
17	Garland Division[8]	0.50	0.21	0.03	0.01	....	2.38	0.33	2.71	0.09	4 400	59	59	28	13	....	38	7	55
	Sun River:																		
18	Fort Shaw Division	0.36	0.04	0.01	....	....	1.54	0.60	2.14	0.23	3 700	58	75	20	5	....	36	26	38
19	Greenfields Division[9]	0.13	0.01	0.06	....	....	1.28	0.65	1.93	0.28	3 700	57	23	75	2	....	31	22	47
20	Umatilla[2]	0.96	0.44	0.06	0.01	....	5.02	0.35	5.37	0.40	470	60	85	1	6	8	32	18	50
21	Uncompahgre	0.92	0.63	0.32	....	....	5.76	0.55	6.31	0.26	5 500	61	47	25	25	3	13	10	77
	Yakima:																		
22	Sunnyside Division	0.60	0.39	0.18	....	....	3.29	0.21	3.50	0.27	800	63	57	7	21	15	23	7	70
23	Tieton Division	0.53	0.35	....	....	....	2.51	0.16	2.67	0.44	1 500	62	44	12	11	33	24	2	74
24	Yuma	0.43	0.28	0.19	0.08	0.07	3.01	0.33	3.34	0.33	120	72	39	3	58	....	14	58	28

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[1] 1924 and 1926 omitted from average on account of water shortages.

[2] Umatilla and Grand Valley data cover years, 1916-25.

[3] King Hill data cover years, 1921-27.

[4] Okanogan data average for 1921, 1923, and 1925, on account of shortages in all other years since 1917.

[5] Orland data omit years of heavy water shortage, 1918, 1920, and 1924.

[6] Rio Grande data cover years, 1919-26.

[7] Shoshone-Frannie Division data cover years, 1922-26.

[8] Losses on water conveyed through Garland Division Main Canals for Frannie Division included in Garland Division losses.

[9] 1919 to 1926 inclusive.

[10] Data are for years, 1917 to 1926, inclusive, except as noted.

Canal and lateral losses do not include losses in canals used solely as reservoir feeders. A case of this type is the Hermiston Division of the Umatilla Project (Table 20), the losses in the Echo Feed Canal being disregarded. Where irrigation deliveries are made both above and below reservoirs served by project canals, as on the Interstate Division of the North Platte Project (Table 12), no attempt to correct for canal losses on water subsequently lost by evaporation and seepage has been made. Reservoir losses by evaporation and seepage are not included in the canal and lateral losses. Where the available data permit, the diversions include—in addition to waters from primary sources of supply, such as rivers and reservoirs—waters diverted from artificial and natural drainage courses within the project, although these may have been included once before with waters diverted from primary sources of supply.

PROJECT NOTES

Special conditions on the projects and their effect in the submitted data may be mentioned as follows.

Belle Fourche Project.—The climate is semi-arid and many farmers irrigate only a part of the area to which water is delivered, thereby making it impossible to determine the actual irrigated areas, except at considerable cost. This practice is gradually being abandoned as increasing irrigation charges necessitate better farming. (See Table 1.)

Boise Project.—A considerable area of this project is dependent on Deerflat Reservoir which is filled through the project main canal when its capacity is not needed for the irrigation of lands above the reservoir. (See Table 2.) The reservoir also receives much waste water and some seepage from the higher lands. For some years records of such inflow have not been obtained. The data presented do not include losses from the main canal, which, from data obtained in earlier years, appear to average approximately 5% of the diversions.

Carlsbad Project.—The storage facilities are inadequate to control stream flow during the non-irrigation season. (See Table 3.) The stream flow during the irrigation season, together with impounded waters, is often less than desired. Pre-season deliveries are, therefore, prevalent.

King Hill Project.—A uniform supply of water is available throughout the season, and while it is inadequate for the irrigation of the entire project, it permits extensive waste at the beginning and end of the irrigation season. (See Table 6.)

Newlands Project.—A limited water supply in 1924 and 1926 caused a material reduction in project waste as compared with earlier years when elimination of such waste was not necessary and the wasting of water was permitted to continue in the interest of superior water service at lowest cost. Recent general increases in farm deliveries are in part traceable to drainage construction. (See Table 11.)

Rio Grande Project.—The apparent great waste of water on this project results from its physical arrangement. (See Table 15.) The project consists of three successive valleys served by six diversions from the Rio Grande. It is customary to run canals at almost maximum capacity in the upper valleys through a large part of the irrigation season with surplus water wasted back to the stream and re-diverted at lower points.

Shoshone Project.—Waters for the Frannie Division (Table 16) are first carried through the main canals of the Garland Division (Table 17), and the resultant losses are included with local losses on the Garland Division. Drainage construction under way on the Garland Division may be expected to result in augmented farm deliveries.

Sun River Project.—The Greenfields Division (Table 19) has a soil and climate adapted to the production of grain by dry-farming methods. Lack of adequate storage reserves has deterred initiation of the collection of construction charges while operation and maintenance charges have been collected only when water was requested and used. The result has been a haphazard farming practice with little reliance on irrigation. With the completion of the Gibson Reservoir now under construction (1929), it is expected that irrigation will be placed on a more stable basis.

Umatilla Project.—The water supply for the East Division of this project, comprising more than one-half the project area, is furnished by Cold Springs Reservoir, which is served by a feed canal from Umatilla River. The indicated canal losses (Table 20) do not include feed-canal losses, which average about 9 per cent.

Yakima Project.—On the Tieton Division (Table 23), the main canal, which is largely concrete flume or concrete-lined, lacks sufficient capacity to meet project demands at the height of the irrigation season. The maximum rate of delivery is, therefore, unduly prolonged. Corresponding data for the Sunnyside Division are given in Table 22.

Yuma Project.—The heavy waste of water on this project (see Table 24) is in part occasioned by the operation of a power plant within the canal system from which the used water is returned to the river when irrigation requirements are small. The main canal is usually operated at near maximum levels in order to minimize silt deposition and the necessity for frequent sluicing. Conservation of water is no object except for occasional short periods when the requirements of the Imperial Valley Canal, diverting below the power-plant return, make the return of all project waste above the Imperial Intake desirable.

The summary compilation in Table 25 gives the average use of water during the past ten years in which water supply was fully adequate. In addition, it classifies the projects as to their predominant soil types. It should not be inferred, however, that the indicated soil type is present to the practical exclusion of others, as, in fact, the soils present in each project in almost every case vary from some form of clay to sand. Were this condition not present, the indicated variation in water use would be even greater than that shown.

SUMMARY

In this compilation, the quantities shown on the individual sheets have been modified in the case of the Belle Fourche, Huntley, Klamath, and Rio Grande Projects to correspond with an under-measurement of 5% in delivery to farms. With these changes, the farm deliveries shown on the summary sheet (Table 25) are then believed to be from zero to 10% less than the actual delivery, and the losses are actually correspondingly less.

The indicated use of water may be considered to represent closely the maximum quantities that the irrigator can use advantageously. Financial considerations have had little influence in engendering a high duty of water because the construction repayment burden on Federal projects is relatively light. On most projects the lands now in cultivation are prone to use waters more freely than will be possible with the project fully developed. Only the Okanogan and Orland Projects have experienced shortages of sufficient severity to influence development. On the whole, it is probable that a higher duty of water prevails on private projects where financial considerations exert a strong influence on the canal and reservoir capacities provided.

Continuous efforts are being made to reduce the excess quantities of water delivered to the farms as well as the transmission losses. However, in practically all cases both excesses are returned to the streams, either as surface run-off or drain discharges, at points such that the unused water is available for re-diversion and beneficial use by other irrigators.

The data indicate project waste and canal losses that may appear surprisingly large to many engineers. Canal losses join the underlying water-table and return to natural channels. Wasted waters so return on the surface. With minor exceptions such return flow is utilized by other projects with a resulting small net loss of waters. Where physical conditions are favorable for project extension, increasing land values in time will foster higher water duties through elimination of waste and reduction in canal losses and farm deliveries. Drainage problems will thereby be minimized and soil fertility conserved with extensive benefits unless carried to the extreme of an insufficient application of water to prevent injurious accumulations of deleterious salts. Reduction in waste, use, and losses will be accompanied by a reduction in return flow with increasing salt content to the detriment of projects that may have become dependent on such supplies.

Much credit is due the various project superintendents for their aid in the preparation of the data and to the Assistant Engineers, C. C. Elder, Assoc. M. Am. Soc. C. E., and Mr. T. R. Smith, to whom much of the work was entrusted and who, by their unflagging interest, have made the completion of this compilation possible. The operation and maintenance of Federal irrigation projects, as well as all engineering and construction work, is under the direction of R. F. Walter, M. Am. Soc. C. E., Chief Engineer of the U. S. Bureau of Reclamation, with headquarters at Denver, Colo.; and all activities of the Bureau are under the general charge of Elwood Mead, M. Am. Soc. C. E., Commissioner, with headquarters at Washington, D. C.

DISCUSSION

Robert A. Monroe,* M. Am. Soc. C. E. (by letter).—This paper should be very useful to engineers engaged in irrigation work since it presents reliable long-term records showing the duty of water for a large area and under wide variation of climatic and soil conditions. Since all these twenty-four Reclamation Service Projects are under one central authority, it is possible to have systematic and complete records kept, and the data presented can be accepted unquestionably as accurate and dependable.

Only two of these projects are located in California: The Klamath Project which lies on the boundary between Oregon and California, and the Orland Project at the upper end of the Sacramento Valley. A great many valuable data along the same lines as those presented by Mr. Debler have been collected by the State of California and are available in published form.†

The Pacific Gas and Electric Company with which the writer was connected in 1929 supplies irrigation water to a large area of land in Placer County, California, and while complete statistical data, such as are presented by Mr. Debler, are not available, it is thought that the following information will be of value in indicating the duty of water in this section.

The area irrigated lies at an elevation of from 300 to 1 500 ft. and is in the rolling foothills in a fan-shaped section of land lying between the American and Bear Rivers. The crops raised consist almost entirely of deciduous fruits; it has been estimated that 99% of the net area irrigated is covered by orchards. The soil can be classified as largely granitic in origin.

The water supply system was originally developed for hydraulic mining operations which were formerly very extensive in this region. After the mines were closed the water was naturally diverted to irrigation use, and with the advent of hydro-electric development between 1890 and 1900 several small plants were constructed to utilize the flow through such head as could be developed cheaply en route. In recent years both storage and conduit capacity have been greatly increased; there is now (1929) a sustained water supply of approximately 350 sec-ft., which is first used for power through a chain of plants under a total head of 3 700 ft., and finally becomes available for irrigation use at the tail-race of the lowest plant.

The storage is located high in the Sierras, where there is a total of more than 150 000 acre-ft. impounded in about 22 reservoirs. There are also 23 regulating reservoirs on the system. The canal system consists of 326 miles of conduit, of which 58 miles may be considered as the main conduit carrying water for both power and irrigation, and the remaining 268 miles are irrigation laterals. The topography is broken, and the irrigation laterals include many long stretches of pressure pipe.

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† *Bulletin No. 6* issued by the Department of Public Works, Division of Engineering and Irrigation, entitled "Irrigation Requirements of California Lands", being Appendix B to a Report to the Legislature of 1923 on the Water Resources of California, contains much information pertaining to the duty of water, and *Bulletin No. 8*, entitled "Cost of Water to Irrigators in California", by Harry F. Blaney, Assoc. M. Am. Soc. C. E., issued by the Department of Public Works, Division of Engineering and Irrigation in 1925, presents data showing the cost of irrigation for nearly every irrigation system of the State.

The gross area that lies under the ditch system approximates 85 000 acres. The area actually irrigated has shown a rather steady annual increase from 13 500 acres in 1914 to 27 100 acres in 1928. Water deliveries for irrigation purposes are made to the consumers at a uniform rate during five months of the year beginning May 1 and ending September 30. No record is available as to the actual quantity of water used each month, as the consumer contracts for a certain number of inches for the season and rotates it on his land as he sees fit. Frequently, several owners who take water from a common lateral will combine to rotate the water on their holdings.

The duty of water is well established and the contracted delivery is almost generally estimated on the basis of 1 miner's in. of water (0.025 sec-ft.) to 5 acres of land, which for uniform flow is 0.3 acre-ft. per acre per month, or 1.5 acre-ft. per acre for the season. The price which the Company receives for this water is $45 per miner's in. per season of 5 months, which makes the cost of water $9 per acre per year, or $6 per acre-ft. No charge is made for water furnished during the non-irrigation season and most of the canals are kept in service the entire year to deliver the small quantity of water necessary for domestic purposes.

The conduit loss was formerly about 45%, but this has been reduced in recent years due to extensive concrete lining and plastering of sections of canals and replacement of wooden flumes with metal; for 1928 this loss was estimated at 37 per cent.

The water supply to date has been ample for the area irrigated, no curtailment having been necessary even in years of extreme drought, such as occurred in 1924, and in most years a substantial quantity is by-passed into the American River. This volume of spill is gradually decreasing as the irrigated area increases, but it seems that the water now available will substantially supply all the irrigable area which is immediately tributary to the system.

IVAN E. HOUK,* M. AM. SOC. C. E. (by letter).—Comprehensive data on the use of irrigation water are needed by all engineers engaged on the design or investigation of irrigation systems. Therefore, the detailed information presented so concisely in this paper, will be a valuable aid to all practicing irrigation engineers in the Western United States.

The data regarding irrigated areas, miles of canals operated, soil conditions, monthly deliveries, character of crops, canal losses, etc., included in Tables 1 to 25, have a pertinent value in showing the conditions under which irrigation is conducted on the various widely scattered projects. However, the most valuable data included in the tables are the total depths of water delivered to the farms plus the depths of rainfall during the growing season. Reference to Table 25 shows that with the exception of the Umatilla Project, of Oregon, the Uncompahgre Project, of Colorado, and the King Hill Project, of Idaho, the average quantities delivered, plus the growing season precipitation, vary from a minimum of 1.56 ft. on the Milk River Project, of Montana, to 4.09 ft. on the Grand Valley Project, of Colorado.

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The corresponding quantities for the Umatilla, Uncompahgre, and King Hill Projects are 5.37, 6.31, and 7.36 ft., respectively. The latter values are relatively high because of the sand and gravelly soils and excessive waste of irrigation water.

The total depths of precipitation during the growing season apparently are not very great as compared with those of irrigation water delivered to the farms plus the depths of growing season precipitation. These quantities may be referred to as "total depths of water applied to the farms", or, simply, "total depths applied". The average values of growing season precipitation recorded in Table 25 are usually less than one-third the total depths applied, the only two exceptions being the Belle Fourche Project, of South Dakota, where the growing season precipitation amounts to approximately 41% of the total depth applied, and the Milk River Project, of Montana, where it amounts to about 58% of the total depth applied. The actual depths of rainfall during the growing season vary from an average of 0.16 ft. on the Tieton Division of the Yakima Project, Washington, to an average of 0.91 ft. on the Milk River Project, Montana.

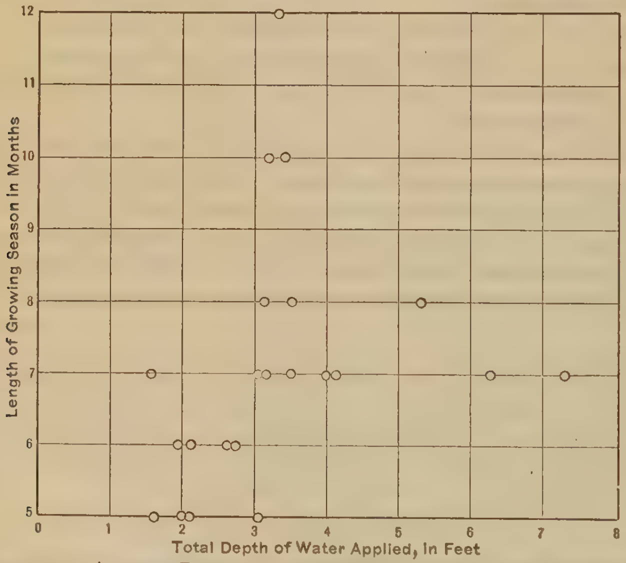

FIG. 2.—RELATION OF AVERAGE DELIVERY OF IRRIGATION WATER, PLUS RAINFALL, TO LENGTH OF GROWING SEASON.

The total depths of water applied to the farms would be expected to vary, in a general way, with the length of the growing season, although, of course, no mathematically definite relation could be anticipated because of variations in soil conditions, character of crops, elevation of project lands, mean annual temperature, and other meteorological conditions, all of which are interrelated in a more or less complex manner. In Fig. 2 the total depths of water applied are plotted as abscissas against the lengths of the growing seasons as ordinates, the lengths of the growing seasons being taken the same as in Tables 1 to 25. Although the plotted points are somewhat scattered, as would be expected, they indicate a general increase in total depth applied with an increase in length of growing season, especially in the case of projects where the length of the growing season does not exceed eight months. In the case of the Rio Grande and Carlsbad Projects, of New Mexico, where the length of the growing season is ten months, and in that of the Yuma Project, of Arizona and California, where it is twelve months, the increased length of the growing season does not require an increased application of water. Apparently, there are two reasons for this: First, a reduction in the quantities of water delivered to the farms during the winter months, when the meteorological conditions are relatively less favorable for plant growth than during the other months of the year; and, second, a reduction in the quantities delivered during some of the summer months, due to a period of dormancy in plant growth caused by the existence of unusually high temperatures. Of course, the points representing the total depths applied on the Umatilla, Uncompahgre, and King Hill Projects, where waste of water and return flow are excessive, fall at the right of the diagram, some distance from the points representing the total depths applied on the other projects.

A question that naturally arises in considering Fig. 2 is as to what proportions of the quantities applied constitute consumptive use. The same conditions that affect the depths of irrigation water delivered to the farms affect the depths of consumptive use, although not necessarily in the same degree. For instance, the mechanical texture of the soil which is probably the most important factor as regards quantity of water delivered, assuming an unlimited supply, has relatively little affect on consumptive use, other things being equal. If accurate information regarding depths of consumptive use were available for the different projects, and if the values were plotted on Fig. 2, the points would probably fall somewhere at the left of those representing total depths applied, the exact position depending on the definition adopted for consumptive use. If consumptive use is defined as the sum of plant transpiration and soil evaporation during the growing season, the consumptive use points would necessarily fall at the left of those representing total depths applied, since some return flow undoubtedly occurs on all projects. However, if consumptive use is defined as the sum of plant transpiration and soil evaporation during the entire year, as would appear to be more reasonable, it is conceivable that some of the consumptive use points might fall at the right of those representing total depths applied, since the total depths of rainfall during the non-growing season, which would be added to the total depth applied during the growing season, in determining the consumptive use, might be greater than the total depth of return flow, which would be subtracted.

In either case the writer would expect the consumptive use points to be more consistently grouped than the points representing total depths applied, assuming, of course, that proper corrections are made for variations in soil storage. Adding the non-growing season precipitation to the total depths applied, as given in Table 25, and leaving out of consideration the three projects on which total deliveries are excessive, gives total yearly depths applied ranging from an average of 1.74 ft. on the Milk River Project, Montana, to an average of 4.57 ft. on the Orland Project, California. These values would be the sums of the consumptive use and the return flow, assuming the variations in ground-water storage to be rendered negligible through the averaging process.

In regard to the depths of consumptive use required for the satisfactory growth of crops, the writer believes that much useful information can be obtained by a study of rainfall and run-off records in Central and Eastern United States.* There are appreciable differences between the Central and Eastern sections of the country and the Western irrigation projects as regards some of the meteorological conditions which affect plant growth, particularly relative humidity and percentage of cloudiness, not to mention rainfall which, of course, is more or less equalized by the delivery of irrigation water. However, it is not believed that the total depths of water required to raise the same crop are materially different in the two sections of the country. Experiments on the water requirements of plants by Montgomery and Kiesselbach, Heinrich, and Sorauer, referred to by Messrs. Lyman J. Briggs and H. L. Shantz, showed that the quantity of water used per pound of dry matter produced was greater in the dryer atmosphere.† However, they also showed that the number of pounds of dry matter produced was greater in the humid atmospheres, so that the total quantities of water used were not so greatly different. Cloudiness seems to reduce the rate of transpiration, but to increase the total water requirement per pound of dry matter produced, although there is some uncertainty in this respect due to the varying rates of photo-synthesis.

Table 26 gives some data on depths of annual consumptive use in the Miami Valley of Southwestern Ohio, for the five years from 1915 to 1919, inclusive, compiled from one of the Technical Reports of the Miami Conservancy District.‡ The consumptive use values were computed from carefully determined isohyetal maps and elaborate stream flow measurements, and, consequently, are believed to be more accurate than most consumptive use data thus far published. The drainage areas represented in the table are more than 90% agricultural land, only comparatively small areas being occupied by cities, towns, farm buildings, roads, rivers, lakes, reservoirs, etc. The length of the growing season, which is approximately the same in all parts of the valley, averages about six months, and paying crops are produced each year. Crops consist primarily of corn, wheat, rye, oats, barley, alfalfa, and tobacco. The acreages of the three classes of crops considered in the author's Tables 1 to 25 are approximately equal and are, therefore, about the same as on the North Platte Project, of Wyoming and Nebraska. The topography of the Miami Valley may be described as gently rolling with the ground elevations of the uplands varying from about 800 ft. above mean sea level near the mouth of the river to about 1 100 ft. near the head-waters.

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* "Evaporation on United States Reclamation Projects," by Ivan E. Houk, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 371.

† "The Water Requirements of Plants," by Lyman J. Briggs and H. L. Shantz, *Bulletin 285*, Pt. II, Bureau of Plant Industry, U. S. Dept. of Agriculture, 1913.

‡ "Rainfall and Run-off in the Miami Valley," by Ivan E. Houk, M. Am. Soc. C. E., Miami Conservancy Dist. Technical Repts., Pt. VIII, pp. 134 to 150, Dayton, Ohio, 1921.

The slopes are comparatively flat near the head-waters but they increase more or less gradually toward the southwest, being comparatively abrupt near the Ohio River. The mean annual temperature of the valley is approximately 53° Fahr., the mean annual relative humidity about 67%, the mean annual rainfall about 37 in., the mean annual run-off about one-third the rainfall, and the mean annual evaporation, or consumptive use, about two-thirds the rainfall.

TABLE 26.—CONSUMPTIVE USE OF WATER IN MIAMI VALLEY, SOUTHWESTERN OHIO.

Gauging station.	Stream.	Drainage area, in square miles.	Mean yearly rainfall, in feet.	Mean yearly run-off, in feet.	TOTAL EVAPORATION,* IN FEET, DURING 12-MONTH PERIODS.†					Average evaporation or consumptive use.
					1915.	1916.	1917.	1918.	1919.	
Sidney............	Miami River.........	555	3.02	0.98	2.35	1.57	2.18	2.17	1.98	2.05
Lockington.......	Loramie Creek......	255	2.98	1.11	....	1.43	1.77	2.08	....	1.88
Piqua..............	Miami River.........	842	3.03	1.05	2.22	1.57	2.07	....	1.91	1.98
Tadmor...........	Miami River.........	1 128	3.08	1.10	2.35	1.62	1.95	2.23	1.79	1.98
Pleasant Hill.....	Stillwater River.....	453	3.15	1.11	....	....	2.06	2.09	1.74	2.04
West Milton	Stillwater River.....	600	3.13	1.13	2.59	1.67	2.07	1.84	1.84	1.99
Buck Creek.......	Buck Creek..........	163	3.22	0.90	2.65	2.28	2.00	2.39	2.22	2.32
Springfield........	Mad River...........	488	3.31	1.07	2.48	2.20	2.05	2.34	2.14	2.24
Wright............	Mad River...........	652	3.29	1.15	2.40	1.95	1.91	2.34	2.10	2.14
Dayton	Miami River.........	2 525	3.15	1.05	2.48	1.72	2.03	2.28	1.95	2.10
Franklin..........	Miami River.........	2 785	3.15	1.05	....	....	1.99	2.33	1.95	2.10
Germantown.....	Twin Creek..........	272	3.19	1.22	2.15	1.67	1.98	2.28	1.80	1.97
Seven Mile	Seven-Mile Creek ...	128	3.29	1.62	2.01	1.30	1.83	1.79	1.42	1.68
Four Mile.........	Four-Mile Creek.....	178	3.27	1.27	2.06	1.23	2.02	2.58	2.06	1.99
Hamilton.........	Miami River.........	3 672	3.17	1.04	2.54	1.75	2.07	2.36	1.92	2.13

* Including interception, evaporation from soil and water surfaces, and plant transpiration.

† Ending September 30.

Rainfall and run-off records were compiled for years ending September 30, that being the time of the year when the quantity of water stored in the ground is least variable. Average yearly totals for the different drainage areas, based on the five years from 1915 to 1919, are included in Table 26. Differences between the yearly rainfall and run-off totals, heretofore usually termed "evaporation", but which are really consumptive use values, are given in Table 26 for each of the five years mentioned. The drainage areas in the table are arranged according to location, beginning at the upper end of the valley. The drainage area of 3 672 sq. miles above Hamilton, the last station in the table and the one farthest down stream, for which rainfall and run-off data were compiled, includes all the other drainage areas listed in Table 26. Consequently, the values of average rainfall, run-off, and consumptive use for Hamilton are really averages for all the other stations in the table. For this reason no final line of average data for all stations has been included. However, a final column of average consumptive use for the five years of record has been added.

The average values of consumptive use are seen to vary from a minimum of 1.68 ft. on the Seven-Mile Creek drainage area to a maximum of 2.32 ft.

on the Buck Creek drainage area, averaging 2.13 ft. for the entire valley above Hamilton. Values for single years vary through slightly greater ranges, but at least a part of the yearly variations are undoubtedly due to variations in ground-water storage, a factor which is believed to be negligible as far as the 5-year averages are concerned. The consumptive use values include plant transpiration, soil evaporation, interception losses, and evaporation from river, lake, and reservoir surfaces, all but the first two being of minor importance. Since the data were compiled for the yearly periods, they include evaporation from soil, snow, and ice surfaces during the winter months, quantities which amount to about 4 in. on the average, or to about 15.5% of the average annual consumptive use in the valley above Hamilton. R. E. Horton, M. Am. Soc. C. E., who has made elaborate studies of interception, estimated that the seasonal interception losses on an average Michigan farm were equivalent to an average depth of 1 in. of rainfall over the entire farm, which would correspond to about 4% of the average consumptive use above Hamilton.* It is interesting to note that the 5-year average consumptive use of 2.13 ft. for the drainage area above Hamilton, is not greatly different from the 9-year average of 2.06 ft. for the same area; and, also, that the 5-year average of 2.10 ft. for the drainage area above Dayton, Ohio, is exactly the same as the 25-year average for the same area.† These comparisons of the 5-year averages with the averages based on longer periods supports the claim that the data in Table 26 have been accurately determined.

George N. Carter,‡ M. Am. Soc. C. E. (by letter).—The author has assembled and presented a vast amount of data gleaned from actual irrigation practice rather than from theory.

To one well acquainted with the project to which any one of the detail twenty-four tabulations may relate, the information is especially valuable. However, to apply these figures to the design of a new project, without intimate knowledge of all the details which affect the demand and use of irrigation water, quite likely might result disastrously. It is just as dangerous to project from such data into the development field as to apply wrongly any highly involved engineering formula in design.

It is to be noted that the use of water varies widely, between the limits of 0.65 acre-ft. per year (the minimum on the Milk River Project, in Montana) and 7.01 acre-ft. per year (the maximum on the King Hill Project, in Idaho). Between these limits are found two projects that are successful and well-balanced in every respect, the Yakima Project (Sunnyside Division), in Washington, and the Boise Project, in Idaho, with an average annual use of 3.29 and 3.60 acre-ft., respectively.

By the expression, "well-balanced," is meant that soil conditions are normal, if there is such a thing as normal soil; that temperature and growing season are conducive to maturing numerous and diversified crops, indigenous to the temperate zones; that no reliance may be placed on rainfall and plant growth is entirely dependent on irrigation; and that the physical irrigation system is well designed and permits of good distribution and control of the water available.

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* "Hydrology of the Great Lakes," by R. E. Horton, M. Am. Soc. C. E., in collaboration with C. E. Grunsky, Past-President, Am. Soc. C. E., Rept. of Engineering Board of Review of the Sanitary District of Chicago, 1927, Pt. III, Appendix II, p. 46.

† *Loc. cit.*, pp. 140 and 142.

‡ Commr. of Reclamation, Boise, Idaho.

Factors which contribute to the wide variations are explained, but it would have been quite helpful if the author could have enlarged upon his description and classification of soils of the various projects. For example, on the King Hill Project, the use of water (see Table 6) greatly exceeds that of other projects and may appear excessive and wasteful to one unacquainted with conditions; but a brief description of the predominating soil characteristic is quite illuminating. The top-soil, averaging from 2 to 3 ft. in depth, is light and sandy, and is underlaid by Snake River black lava sand to great depths. The water-consuming possibilities of this soil and subsoil are obvious; likewise is the problem that such a condition imposes upon the farmer. Irrigations must be very frequent. The writer examined a field of alfalfa seeding on the King Hill Project, planted with a nurse crop of wheat, and found the young alfalfa plants wilting six days after thorough irrigation. Furthermore, the application of such large quantities of water is rapidly leaching all plant food elements out of the soil, as demonstrated by the fact that each year the use of water is increasing, while crop yields are gradually decreasing.

The methods of securing field data and reducing them to as accurate forms as possible, are in keeping with good practice. The Twin Falls North Side Land and Water Company while constructing a 185 000-acre Carey Act irrigation project in Idaho, carried on for five years what was perhaps as comprehensive and thorough a study of the use of water for irrigation as has ever been undertaken in the United States. The under-measurement of water delivered to the farmer was found to be 5%, occasioned in large part by the tendency of the Cipolleti weir to under-read due to velocity of approach, caused by silting of weir pools. It was found more satisfactory to calibrate the weirs in their actual operating condition, employing expert hydrographers for the purpose, than to attempt to maintain them in ideal physical condition.

The author finds the expression "loss in feet of depth per day" over exposed water surface or wetted perimeter "impractical." It occurs to the writer that this is a most rational measure of canal loss. Expressed on this basis, a unit is afforded which may be applied to any operating condition between peak load and no load. A canal system may carry full load with a net delivery of 70% and half load with net delivery of only 40 or 50 per cent. Under these two conditions, the loss in feet of depth per day will be practically the same, while the percentage of loss varies greatly. Wide, shallow, low-velocity canals will have a much higher percentage loss than when the same quantity of water is conveyed in a deeper and higher-velocity channel, while the loss in depth of feet per day over wetted perimeter of exposed water surface is the same for either type of canal, all other conditions being equal.

Some elaboration might well be made as to the quantities of waste expressed as "Percentages of Total Diversions for Seasons", in Tables 1 to 24. An efficiently operated canal system should not waste out of the lower end much more than 4 or 5% of the total supply. Through careful operation, this figure is often reduced to 2 or 3 per cent. Any waste of more than 4% is ordinarily avoidable. Therefore, in using the author's data on this item, it may be understood that the large percentages are voluntary and need not be admitted as a necessary part of the available supply.

The collection and intelligent application of data such as the author has presented, are invaluable in a consideration of any irrigation project, especially a large one. Owners and operators quite frequently under-estimate the value of such a survey and hesitate to appropriate funds for the purpose. Nevertheless, the characteristics of the various parts of the water-distribution system, with respect to the use of water, as revealed by detail hydrographic records, will locate places of excessive loss and point the way to a remedy, thus saving many times the cost of such work.

Given favorable soil and climatic conditions, the prosperity of an irrigation project is reflected almost entirely by the quantity of water it has available. Despite extravagant and wasteful use, water-logging of land, and other difficulties ordinarily credited to over-use of water, the project's well-being and financial success will be in proportion to the quantity of water used. Land values, diversity and intensity of farming, and crop yields, all go down in proportion as the available water supply diminishes, and almost without exception the greatest use of water is found to be coupled with the most prosperous project.

The popular and numerous investigations undertaken throughout the irrigated West about twenty years ago to determine the duty of water have been harmful. They were predicated on laboratory methods rather than actual field conditions, and they aimed at a result which would be the minimum, in order that the irrigated acreage of the Western States might be extended to a maximum. Knowledge of the conditions under which the farmer would perpetually be compelled to till these extended acres was lacking. The experience of the last twenty years of irrigation activities has well demonstrated the fallacy of the original object.

In the beginning of irrigation on a large scale throughout the Western States it seemed desirable to produce a maximum of crop per unit of water, but now it is evident that the maximum per unit of land is the only basis upon which the irrigation farmer can remain. Otherwise, taxes, overhead, labor, and equipment costs, carry the farmer to the point of "diminishing return" and frequently beyond it.

Witness the voluntary efforts and expenditures of many co-operative irrigation companies to enlarge their water supplies after they have received control of their projects from the builders. They have learned, during the years of pioneering, that only with a generous and dependable water supply can they expect to prosper.

RALPH L. PARSHALL,* ASSOC. M. AM. SOC. C. E. (by letter).—In presenting this paper with its official records of the use of water on Federal irrigation projects, the author has made available data of great value to irrigation engineers. Because of its importance, however, it is thought that he should have gone more into detail in explaining the underlying causes for the large percentage of the total diversion that is returned to the parent stream, since a study of the data presented might lead to the belief that project waste and canal losses are much greater than should have been expected; and to those not thoroughly familiar with irrigation requirements, it might appear that such wastage showed inefficient operation.

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In practically all the irrigated areas in the West, the matter of sand and silt carried into both main canals and laterals is of grave concern from the standpoint of operation. It is the usual practice to provide one or more wastegates at suitable points along the main canals and, through these openings, to flush out the accumulated deposits. The water used for sluicing purposes is, in many cases, a large proportion of the total diversion. Thus, Table 25 indicates marked wastage for the Rio Grande and Yuma Projects. These items are explained by the author as necessary in supplying other areas at lower elevations and providing water for hydro-electric power. Such expenditures in water might not be looked upon as waste, since they are put to a useful purpose. The use of the word, "waste", is not in accordance with the general meaning, which is "loss without equivalent gain." In this restricted sense, the writer is of the opinion that Table 25 might be misleading to some in presenting these large percentages. With the irrigation canals diverting from the Arkansas River in Colorado, for example, it is the usual practice to sluice occasionally for the purpose of scouring out the accumulated deposits. In some cases this is done each day for a period of 1 hour, an actual waste of water to the users amounting to about 4 per cent. In no case would this waste exceed 10 per cent. The water used in sluicing constitutes a direct loss to the canal. Another fruitful source of waste occurs through leaky farm and lateral head-gates, as well as from poor structures, such as flumes, siphons, or other artificial channels.

The canal and lateral losses appear to be large, especially for some of the projects having large percentages of lined canal or pipe lines. In the case of the Umatilla Project, the miles of pipe and lined section together approximate the total miles of canals and laterals; yet the losses are recorded as being not greatly different in percentage from those on other projects with a much smaller proportion of lined channel and with similar soil conditions. Concrete-lined canals are not entirely immune from seepage losses.

The measurement of water from the standpoint of duty requires a fair degree of accuracy. Because of the great variety of conditions and limitations imposed in the measurement, the author assumes that the actual amounts measured are somewhat less than 10% smaller than those stated in the tables. Unless the conditions are extremely adverse, the writer believes the deviation ought not to exceed 5%, provided suitable devices are used in the measurement. The calibrated rating flume is usually a poor means of securing dependable measurements. The filling and scouring of the channel and the accumulation of aquatic growth, all tend materially to affect the accuracy of this type of device. The rating flume, if only occasionally checked for performance, may introduce considerable error in the final result. In some cases constant attention to check gaugings is necessary to determine the daily rate of change in the calibration. The necessary hydrographic expense of keeping abreast of such conditions may not be warranted because of allowable limits assumed in the management or administration of the water supply. Because of aquatic growth some Colorado canals equipped with the ordinary rectangular rating flume have changed between June and September as much as 75% in the calibration. Infrequent check gaugings, in such cases, would involve considerable error in estimating discharge. In many parts of the irrigated West, where the value of water is becoming increasingly greater, better and more dependable measurements of flow are much needed.

The use of the standard weir for the measurement of farm deliveries is generally unsatisfactory. For correct measurements the weir must be provided with standard distances, crest to floor and side of weir to bank or wall. Weirs without end contractions eliminate one of these sources of error. Almost without exception, in the case of the farmer's weir, the pool or basin up stream is found to be filled with sand or silt nearly or actually to the crest line. This tends to a greater delivery than is apparent from the indicated head or depth over the crest. The farmer is scientific enough to know that this defect is advantageous to him, and is, therefore, not overly ambitious to maintain standard conditions.

Submerged weirs and orifices are also subject to a varying degree of accuracy; however, in such cases, the errors will be compensating. Ordinarily, in farm practice, no instrument record is maintained of the variation in the upper and lower heads, where a change in either or both results in a corresponding change in the flow. Thus, in using a submerged device, the chance of error is greater than in the single head measurement. A standard weir submerged to about 70% will reduce the rate of discharge approximately 35 per cent. In endeavoring to obviate the double-head submerged flow over weirs or through orifices, it is found that the improved Venturi flume, when operating at about 70% submerged flow, will be discharging according to the free-flow law and thus only involves a single head. In this respect it is believed that this type of device is superior to other commonly used methods of measurement.

In the study of the duty of water, there is a question as to whether or not it is proper to add the total precipitation over a period of months to the quantity or depth of water applied as irrigation to represent the acre-feet per acre application to the crop. When storms occur in moderation and the soil can absorb the precipitation, such depth of rainfall can be legitimately charged. For an intense downpour, when the top soil is dry, it has been observed that the surface becomes packed or crusted, due to the impact, thus causing a considerable run-off. It is further known that, for showers of 0.1 to 0.2 in., the penetration is not sufficient to be of any benefit from the standpoint of adding moisture to the soil; in fact, quite the contrary is true. Observations on soil moisture after a moderate shower show a depletion of moisture instead of an increase. The question as to what amount of the total rainfall can be justly accredited to useful soil moisture has not been determined, the soil itself being an important factor. It is logical, therefore, to assume that to give full weight to the total rainfall is misleading. The question deserves thorough study.

E. B. Debler,* M. Am. Soc. C. E. (by letter).—Interesting data are presented by Mr. Monroe on a class of projects which is quite common in California, but is rarely found outside that State. California has large areas characterized by deep finely textured soils of unusual water-holding capacity. The winter rains are stored in these soils, and, to a large degree, furnish the water needs in the succeeding growing season with a resulting high duty of irrigation water. Furthermore, these lands are in nearly every case adapted to fruit culture and so escape the need for the liberal water supply required for the growing alfalfa and similar crops of high water requirement which must necessarily be the backbone of irrigation in the inter-mountain States.

Mr. Houk graphically presents the relation of depth of water applied to length of growing season. The most important feature controlling application of water is the mechanical texture of the soil and subsoil. Soils of great porosity require heavy applications wherever general farming is practiced. On the Orland Irrigation Project, in California, annual applications of water vary from 2 to 6 ft. per acre, and in the Snake River Valley, in Idaho, from 2 to 10 ft. per acre. The lack of fertility which is so characteristic of most soils of extreme porosity precludes, in individual cases, the production of fruits in the one area and of sugar beets in the other, as these crops are seldom profitable except in fertile soils of considerable depth.

The effects of continual application of large quantities of water in the reduction of soil fertility has been mentioned by Mr. Carter and has been noticed in many localities on the Federal projects. The only way of escape where this is the case appears to be the adoption of crops requiring intensive cultivation and permitting a comparatively heavy expenditure in the control of water so that application will not exceed greatly the water-holding capacity of the surface soils. This course, however, can be adopted only to a limited extent in that a very large part of the irrigated area is not adapted to the production of such crops by reason of unfavorable climate and marketing difficulties.

Mr. Houk has presented much data bearing on consumptive use, a subject even more far-reaching than that of the delivery of water, to which the paper applies. Where development is governed by water supply the undeveloped resources in the way of non-irrigated lands will lead to a study of the ways and means to reduce consumptive uses to a minimum for each unit of crops that can be produced profitably.

Mr. Carter suggests that canal losses might well be estimated from exposed water surface. The U. S. Bureau of Reclamation conducted experiments for a number of years on constructed canals in order to ascertain the coefficients of loss, in feet per day, over the wetted area. The loss through soils was found to vary from amounts as low as 0.25 ft. to more than 1 ft. of depth per day. Over the same section of canal the losses were also found to vary greatly from one year to another. For a type of soil given the same description by different engineers, the range in losses reported varied between very wide limits. A part of the difference is due no doubt to variance in definitions of soils by different men. In order to obtain a uniform definition it would be necessary to make a mechanical analysis of the soils; but it is also well known that mechanical properties alone will not tell the entire story, since the nature and distribution of soil chemicals and humus are very important factors.

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* Engr., U. S. Bureau of Reclamation, Denver, Colo.

The depth to ground-water is important because, with a very high water-table there is a back pressure from the water-table to the canal. Where silty waters are carried, an impervious skin coat forms on the canal banks and the leakage is determined, not by the soils through which the canal passes, but by the thickness and character of the deposit of silt. Where it is customary to remove the silt from a canal at the end of each irrigation season there may be a very great variation in losses through the season. On the other hand, if a canal is originally built with excess capacity, it may be the practice in operation to leave a large part of the silt coat undisturbed.

Aside from interpreting the data obtained on operating systems, many assumptions are necessary in applying such formulas as might be evolved to the conditions that will obtain on a system proposed for construction. The combined result of all these features is that the observed losses cannot practicably be converted into formulas or coefficients that can be applied in designing a new canal system. On the whole, it has been found more practicable to make a direct estimate of the percentage of loss on new projects, with such percentage greatest at the end of the irrigation season. This, however, can be only considered a practical application of the theory that losses are largely dependent on the wetted areas which do not differ materially through the irrigation season. Except where conservation of water is of no interest, losses on the average project do not vary greatly from 35 to 40% of the waters diverted. Where a reduction in losses becomes desirable, either for the prevention of seepage or on account of limited stream supply in comparison with area, leading to the adoption of canal lining, the loss may be materially less than the percentages indicated.

Mr. Carter mentions the desirability of a description of classification of soils on the various projects. This matter was given consideration, but it was found that nearly every one of the projects has soils and subsoils ranging from heavy to loose and that two observers will seldom give the same classification for any one kind of soil. It would be difficult to convey to the reader an accurate picture of soil and subsoil types. While soil is the most important factor in determining the water duty, it would also be very desirable to have full descriptions of irrigation methods, crops, liberality of water supply, etc. Impracticability of precise descriptions for these factors within the limits of a paper of moderate length, precludes their discussion.

Attention has been directed to the apparent large waste. Contrary to the understanding of Mr. Parshall, the indicated data do not include sluicing waste at the point of diversion. On some projects stream flow far exceeds all irrigation requirements and excess waters are carried through a large part of the season for greater ease in operation, with the irrigator permitted to receive and reject water practically at will. A similar condition prevails on projects where stream flow is merely passed through project canals for the use of other areas. On projects where conservation of water and canal capacity are important, waste is not heavy except in years of surplus supply. The Boise Project, with an average waste of 2%, is of this type.

Mr. Parshall appears to have gained the impression that the arbitrary correction in recorded deliveries was made to offset estimated errors in measurement. This, however, was not the case. Measurements have not been disturbed. The increases were made to offset deliberate practice in the field of reporting less delivery than is actually made, in the interest of improved business relations between the farmer and the ditch-rider.

The magnitude of precipitation during the growing season has been found to be very material in fixing irrigation requirement. Along the eastern slope of the Rocky Mountains, growing season precipitation averages 10 in., while in the Great Basin, it averages 2 in. The latter, and possibly an equal part of the former, occurs largely in small showers without benefit to crops except as relative humidity is temporarily increased. The remainder of the larger amount materially reduces the irrigation requirement. In fact, on some projects, a little farther east, in the so-called "semi-arid" belt with normally 12 to 15 in. of rainfall in the growing season, irrigation is considered an adjunct to farming rather than the mainstay, and is so practiced.

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Paper No. 1756

WATER SUPPLY FROM RAINFALL ON VALLEY FLOORS*

By A. L. Sonderegger,† M. Am. Soc. C. E.

With Discussion by Messrs. Harold C. Troxell, Harry F. Blaney, C. W. Sopp, W. P. Rowe, Ivan E. Houk, Charles H. Lee, and A. L. Sonderegger.

Synopsis

This paper presents an analysis of the phenomenon of rainfall penetration on valley floors in semi-arid areas and a discussion of methods for the quantitative determination of the resulting water supply, with special reference to conditions in Southern California.

The following specific examples are discussed:

1.—Determination of rainfall penetration from soil moisture tests and from springs in the Murrietta-Temecula Area.

2.—Determination of penetration on valley floors by comparison with mountain run-off in San Bernardino Valley.

3.—Determination of penetration from rainfall and irrigation rise of water in test wells in Pauba Valley.

Introduction

In the semi-arid West, particularly in the valleys of Southern California where seasonal rainfall varies from 6 to 24 in., the water supply resulting from deep penetration of rainfall on the valley floors is of great economic importance.

Note.—The Special Committee on Irrigation Hydraulics has selected the subject of "Evaporation from Soils" as one of ten for study and research. This paper was submitted to the Committee by its author, and the Committee has recommended its publication (see Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1929, Society Affairs, p. 96).

* Published in May, 1929, *Proceedings*.

† Cons. Engr., Los Angeles, Calif.

In the past, engineers have sometimes under-estimated this source of water supply, while in the people's mind the safe yield of ground-water sources has been confused with the capacity of the basins and the supply has been considered unlimited. Recent investigations would indicate that under favorable conditions of rainfall and soil formation the permanent recharge of ground-water basins from this source may amount to as much as 1 acre-ft. per acre in one season and during a period of years to as much as the supply from the tributary mountain water-shed.

The disposal of rainfall is as follows: (1) Storm run-off; (2) evaporation losses; (3) transpiration losses; and (4) deep penetration. Items (2), (3), and (4) may be grouped under the general term, "absorption", for comparison with Item (1). With closed basins, the resulting water supply represented by Items (1) and (4) may be determined by direct measurement (making allowance for consumptive use and evaporation losses from moist areas); while with open valleys indirect methods may have to be resorted to by assigning values to the different factors. Measurements and other information leading to the determination of any of the factors are, therefore, of general interest. Certain studies made for the alluvial valleys of Southern California, the methods applied, and here presented, may be useful to other investigators.

General Conditions Affecting Penetration

Among the general conditions that affect penetration may be listed the character of alluvial deposits, the topography of the ground, the use made of the soil for purposes of agriculture, and the intensity of storms.

Character of Alluvial Deposits.—Seepage water that penetrates intermittently below the limits of evapo-transpiration activity must ultimately cause the wetting—to field capacity—either of the entire formation overlying the water-table, or of well-defined ducts reaching from the surface to the water-table. If there is additional seepage from above, there must be a corresponding addition to the ground-water body.

Alluvial deposits are essentially heterogeneous in formation so that the percolating fluid will naturally find lines of least resistance. Therefore, a uniform saturation over large areas is not likely to occur, but rather the formation of more or less defined ducts. When percolating water strikes an impervious stratum, movement downward is interrupted and a suspended water-table is formed. Gradually, lateral percolation and a circuitous route for the percolating fluid are then established until another pervious deposit again permits a more or less vertical movement. Whenever a suspended water-table has been formed, the drainage from it will occur over the edges of the impervious stratum and is likely to collect into trickling streams, as illustrated in Fig. 1. These conclusions are supported by the fact that during the process of excavating shafts in alluvial fills it is quite common to strike dry materials alternating with moist materials and occasional streams of water. Inasmuch as alluvial deposits almost invariably show an alternation of pervious and impervious strata, the phenomenon indicated in Fig. 1 is probably that most commonly encountered. Under such conditions, only a part of the alluvial mass between the surface and the ground-water table will be saturated.

The recharge of the ground-water may be hastened or delayed by impervious strata. Large clay bodies may not only cause a regulation of an irregular water supply, but they may also provide storage of a magnitude not appreciated because it is not readily traced.

Absorption is favored by the wedging action of roots, especially decayed roots, which have a tendency to drain a saturated mass along defined canals.

It is concluded, therefore, that both the heterogeneous character of alluvial soils and the action of roots tend to concentrate percolation along lines of least resistance and that a uniform wetting of the soil over large areas to the depth of the root zone is not likely to occur.

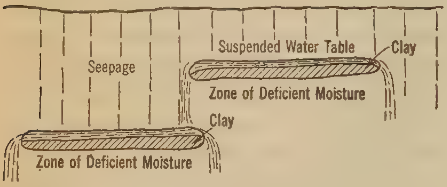

Fig. 1.—Effect of Impervious Strata on Disposal of Seepage.

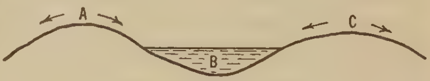

Fig. 2.—Concentration of Rain Water Due to Topographical Conditions.

Topographical Conditions.—The effect of a rainstorm is intensified by irregularities of topography, whether they are the gentle undulations which occur on an apparently true plane, or hog-wallows and hummocky undulations, as in Fig. 2. It is apparent that some of the rain falling on Points *A* and *C* will flow on the surface to the depression, *B*. A 3-in. rain may produce at *B* a depth of water of as much as 6 in. and a seepage greatly in excess of that at Points *A* and *C*. This concentration of rain water, therefore, induces a corresponding concentration of seepage and may be responsible, in part, for deep penetration when the seasonal rainfall, uniformly absorbed, is not sufficient to satisfy the moisture deficiency and evapo-transpiration losses.

Intensity of Storms.—The all-important factor appears to be the intensity of storms; that is, rain falling in consecutive days, or within intervals not exceeding two days. Generally speaking, the number and intensity of storms vary with seasonal rainfall. Dry years with rainfall as great as 12 in. are generally devoid of heavy storms and seldom have a storm of more than 3 in. Very wet years generally produce one storm of outstanding intensity and a number of minor storms, although there are exceptions to this rule. Inasmuch as the rate of evaporation is a maximum immediately after a rain, and since even showers foster the growth of grasses, the light rains of a dry winter are, as a rule, immediately wasted, while on the other hand, the lighter storms of a wet winter may saturate the soil to field capacity, and subsequent heavy storms may produce nearly 100% penetration. The superior effects of one heavy storm over a number of light ones becomes magnified when the soil is non-uniform.

Fig. 3 shows the distribution of rainfall into storms of varying amounts based on seasonal and daily records of four stations in the Murrietta-Temecula Area (Riverside County), of Southern California. This is an undulating area, ranging in altitude from 1 000 to 1 600 ft., with isolated peaks reaching elevations of 3 000 ft. It is an interior valley and, comparatively speaking, it is arid (Fig. 4). Seasonal rainfall records are available for a number of stations in the valley area, indicating a long-year mean of 14.5 in., the minima approaching 6 in. and the maxima, 28 in. The distribution of storms for the station at Wildomar (Elevation 1 254), for three characteristic years, is listed in Table 1.

TABLE 1.—RECORD OF STORMS AT WILDOMAR, CALIF.

Season.		Seasonal rainfall, in inches.	Total, 2 in., or less.	2 in. to 4 in.*	4 in. to 6 in.*	6 in., or more.*
Year.	Description.					
1923–24	Dry	7.37	4.41	2.96	....	.....
1916–17	Normal	14.84	6.76	....	....	8.08
1921–22	Wet	27.89	4.61	2.00 2.73 2.55	4.5	11.46

* One storm each.

The important feature of this analysis is the illustration of the occurrence each season of one storm of outstanding intensity, necessitating the insertion of a special curve of "maximum storms". Example 1 explains the curves in Fig. 3.

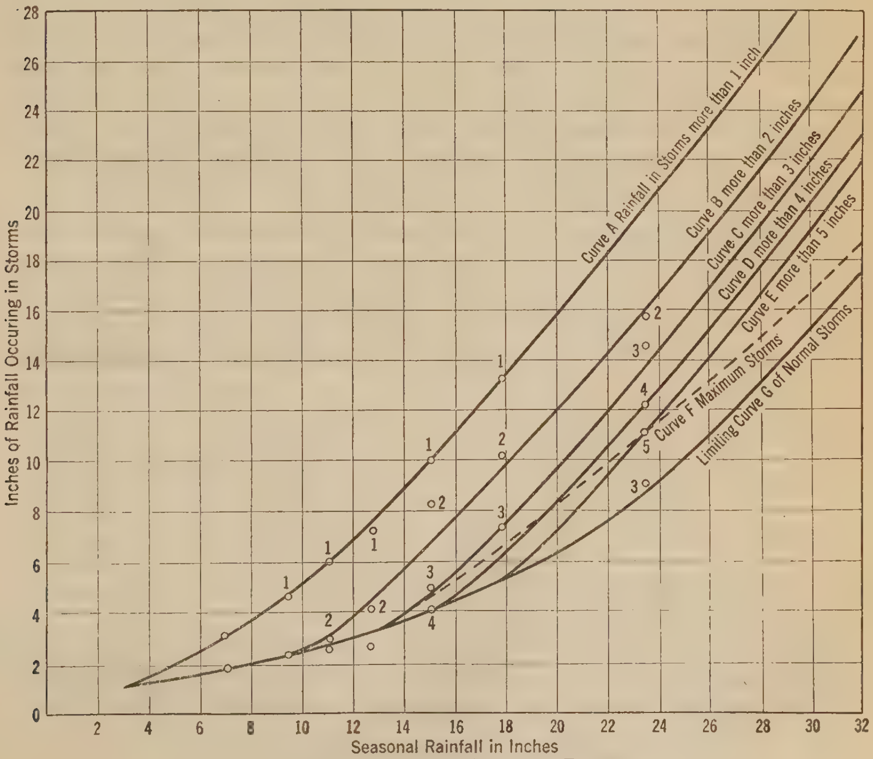

FIG. 3.—DISTRIBUTION OF SEASONAL RAINFALL.

Example 1.—

Given a seasonal rainfall of	22.0 in.
From Curve *B*, the rain falling in storms of more than 2 in., is	14.4 “
Therefore, rainfall in storms of less than 2 in., is	7.6 in.
From Curve *C*, the rain falling in storms of more than 3 in., is	12.0 in.
From Curve *F*, the possible maximum rain storm is	9.9 “
Therefore, the rainfall in storms of more than 3 in. and below maximum, is	2.1 in.

From this it may be concluded that, for a seasonal rainfall of 22 in., there may occur:

One storm of	9.9 in.
Storms from 2 in. to 3 in. (14.4 — 9.9)	4.5 “
Storms of less than 2 in.	7.6 “
Total	22.0 in.

The effect of a series of dry years is to deplete the moisture content to the depth of plant roots, or possibly below that point, evaporation being the minor and transpiration the major factor. It may require a very wet year to recharge a deeply depleted soil. However, if further wet years follow, the amount of deep penetration must assume substantial proportions. This is inferred from a segregation, into storms, of the rainfall at Wildomar (in the arid Murrietta Valley) for three consecutive wet years (see Table 2).

TABLE 2.—Analysis of Rainfall at Wildomar, Calif.

Season.	Seasonal rainfall, in inches.	Storms of 2 Inches, or More.		
		One each, in inches.	Total, in inches.	Percentage of seasonal rainfall.
1913–14	21.33	3.14 5.50 6.81	15.45	72
1914–15	22.65	2.73 2.00 2.58 7.89	15.20	67
1915–16	21.06	2.20 5.15 9.33	16.68	80

It is concluded that the major part of deep penetration and recharge from rainfall occurs during a series of consecutive wet years and that the magnitude of the maximum storms may be considered a ruling factor affecting penetration.

Consumptive Use of Cover.—The investigators of the U. S. Department of Agriculture have published the results of great numbers of experiments relative to evaporation and transpiration losses for different types of soils and crops. On the basis of these findings, values for the determination of deep penetration may be assigned to moisture deficiency at the beginning of a rainy season, and to the losses by evaporation, transpiration, and run-off in connection with a study of the individual storms of a season. This method, of necessity, is based on the assumption that rain will remain uniformly distributed over the area after it falls; that the moisture depletion at the end of the season is more or less uniform over the area in question; and that the soil is of uniform texture to the depth to which such depletion occurred. These assumptions, in all probability, will result in excessive values for losses and low values for penetration. On the other hand, if based on daily rainfall records and carried through an entire season, this method will take into account the character of individual storms in a manner no other method will permit, so that the results must be considered as reliable, although too low. Each season, however, must be studied separately.

Harry F. Blaney, Assoc. M. Am. Soc. C. E., has estimated* the total average transpiration for all active growing vegetation in the northern valleys of Southern California for the six winter months (rainy season) as 1 acre-in. per acre per month. For bare lands, vineyards, and deciduous orchards which are clean cultivated, no transpiration losses occur in the winter. Furthermore, the deficiency of soil moisture at the end of the summer is, for various crops:

Irrigated grain and citrus............	2 to 4	acre-in. per acre
Non-irrigated grain, grass, and weeds.	4 to 6	" " " "
Deciduous, grapes, walnuts, peaches..	6 to 8	" " " "
Brush	6 to 8	" " " "

Evaporation losses are 0.5 in. per acre after each rainstorm, or from 4 in. to 8 in. per season. From these data the water supply that may be required to satisfy the moisture deficiency at the beginning of the rainy season and the evapo-transpiration losses during the same time, is estimated by Mr. Blaney as follows, depending on the wetness of the preceding years and the type of soil:

Bare land....................................	6 to 10 in.
Irrigated grain and citrus.........................	12 to 16 in.
Non-irrigated grain and weeds..................	12 to 18 in.
Deciduous orchards, grapes......................	10 to 16 in.
Brush	12 to 18 in.

DETERMINATION OF PENETRATION FROM SOIL MOISTURE TESTS ON NORTH MESA, MURRIETTA-TEMECULA AREA

Penetration from rainfall may be ascertained, both as to quantity and depth, by soil moisture tests made before and after a wetting. Test samples are obtained by soil auger or soil tube, and the holes of successive tests as a rule are within a few inches of each other. The test holes are then plugged by tamped earth. For fairly uniform soils the results may be considered accurate for the area in the immediate vicinity of the test holes and sufficiently reliable for general conclusions regarding larger areas. In many citrus orchards of Southern California, soil moisture tests are a matter of routine, made to determine the immediate irrigation needs.

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* *Bulletin 19*, "Santa Ana River Investigation," State of California, Dept. of Public Works, Div. of Eng. and Irrig., 1929.

TABLE 3.—SEASONAL RAINFALL AND STORM RECORD FOR THE MURRIETTA-TEMECULA AREA, OBSERVED AT WILDOMAR, CALIF.

Season.	Seasonal rainfall, in inches.	Storms of more than 2 in.
1913–14	21.33	3.14* 5.50 6.81
1914–15	22.65	2.73 2.58 2.00 7.89
1915–16	21.06	2.20 5.15 9.33
1916–17	12.26	2.75 2.51
1917–18	10.94	3.34
1918–19	7.10	.....
1919–20	11.76	2.64 2.33
1920–21	10.10	2.45
1921–22	27.89	2.00 2.73 2.55 4.54 11.46
1922–23	8.37	.....
1923–24	7.37	2.96
1924–25	7.46	.....
1925–26	14.84	8.08
1926–27	20.89	3.08 10.90

* One storm.

Some irrigation and rainfall penetration tests were made* in February and March, 1927, on the North Mesa of the Murrietta-Temecula Area in Riverside County, Southern California. Fig. 4 is a map of this area. These experiments were made in conjunction with the determination of the water supply available from the rainfall which was indicated by springs in the same general locality. The region is comparatively arid and the soil is an ancient, rather compact alluvium, a combination which would lead the casual observer to the conclusion that no deep penetration could occur. Table 3 gives the seasonal rainfall and storms for a period of years for the station at Wildomar in Murrietta Valley. The altitude of Test Plot No. 1 (see Fig. 4), is 1 400 ft. and the average seasonal rainfall at that place is about 14.5 in. Water was applied by irrigation and by rainfall. Moisture determinations were made from soil-auger samples and were averaged from observations on three holes. The soil is a sandy clay loam, or silt loam, with strata of pure sand at depths of 12 to 15 ft. After a rain, the mesa shows many shallow puddles, from which the water seeps away or evaporates. The land had been dry farmed. The water-table stands at a depth of about 200 ft. The size of test plots was 20 by 20 ft. They were banked on the edges and flooded.

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* By Willard G. Babcock, Agricultural Chemist, Riverside, Calif.

The values for Table 4 were obtained from undisturbed cores of soil taken on March 16, 1927. The moisture equivalent test was made to obtain a value for the "specific retention", which is a term used to express the quantity of water which a soil or rock will retain against the pull of gravity if it is drained after having been saturated.*

TABLE 4.—CHARACTERISTICS OF SOIL IN TEST PLOT NO. 1.

Factor.	DEPTH BELOW SURFACE:	
	2 ft.	4 ft.
Specific gravity (volume weight)	1.57	1.81
Wilting coefficient	5.8	6.0
Moisture equivalent	10.7	11.0
Porosity, in percentage by volume	37.2	29.0
Probable yield, in percentage by volume	20.3	9.1

The moisture equivalent† is equal to $100 \frac{c}{W}$, in which, c is the weight of the water that remains in the soil after it has been saturated and then subjected to a centrifugal force 1 000 times the force of the gravity, and *W* is the weight of the soil when dry. It is an arbitrary ratio used in indirect determinations of the hygroscopic and wilting coefficients of soils and may also be found useful for estimating the specific retention.‡

The approximate limits are given by Meinzer§ as follows:

Coarse sand	1.5 to 2.5%
Fine sand	2.5 to 7.5%
Sandy loam	7.5 to 24%
Clay loam	24 to 31%

Quoting from a report‖ by O. W. Israelsen and F. L. West, Associate Members, Am. Soc. C. E.,

> "Correlation between the moisture equivalent and the maximum amounts of water found after irrigation show a gratifying agreement and suggest that the moisture equivalent might be made a basis of judging maximum capillary capacities (essentially specific retention)."

In other words, the moisture equivalent is approximately the field capacity of the soil. The wilting coefficient, as defined by Mr. L. J. Briggs, is equal to the moisture equivalent divided by 1.84.

Table 5 shows the moisture equivalent from determinations made on the test plot at North Mesa (see Fig. 4) during February and March, 1927. Table 6 contains a record of the quantity of water applied on the test plot. Assuming an average specific gravity of 1.69 for the dry soil, 1% of moisture is equivalent

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* *Water Supply Paper No. 489,* U. S. Geological Survey.

† *Water Supply Paper No. 494,* U. S. Geological Survey, Sheet 12.

‡ *Loc. cit.,* Sheet 11.

§ *Water Supply Paper No. 494,* U. S. Geological Survey.

‖ *Bulletin No. 183,* Utah Agricultural Experiment Station.

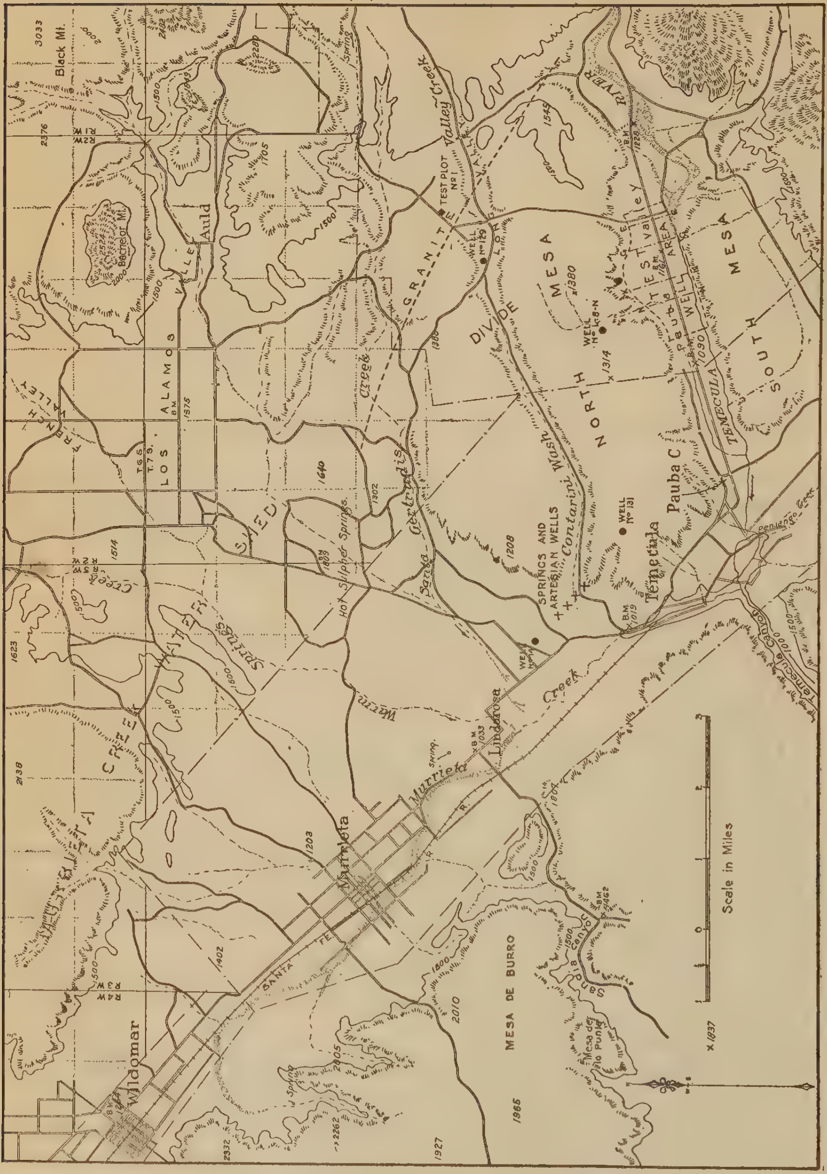
Black Mt.
3033
2376
Bachelor Mt.
2524
R2W.
R1W.
R3W.
R4W.
FRENCH VALLEY
LOS ALAMOS VALLEY
Auld
T.6S.
T.7S.
B.M. 1375
1514
1623
2138
1640
1302
Hot Sulpher Springs
Santa Gertrudis Creek
GRANITE
TEST PLOT No 1
Valley Creek
DIVIDE
NORTH MESA
SOUTH MESA
WELL No 129
WELL No 131
Contarini Wash
SPRINGS AND ARTESIAN WELLS
Temecula
Pauba C
TEMECULA
TEMECULA RIVER
Pechanga Creek
Temecula Canyon
B.M. 1019
B.M. 1033
Lindarose
Murrieta Creek
Murrieta
Warm Springs
SANTA FE R.R.
1203
1402
Wildomar
MESA DE BURRO
Mesa de la Punta
San Diego Canyon
1865
1927
2010
2332
2262
1837
Scale in Miles
0
1
2

FIG. 4.—MAP OF MURRIETTA-TEMECULA REGION, CALIFORNIA.

to 0.2028 surface in. of water. The values given in Column (2), Table 5, indicate a heterogeneous soil, the thirteenth foot being sandy, the top foot, a sandy loam, and the fifteenth, a loam. From Column (2), Table 5, the total field capacity for the first 9 ft. is computed at 20.16 surface in. of water, or a column of water, 20.16 in. high. The total moisture content on February 4, for the same 9 ft. is obtained from Column (3), Table 5, and computed at 8.66 in. Consequently, there was a moisture deficiency in the first 9 ft. of 20.16 — 8.66, or 11.50 surface in., which had to be satisfied before penetration below the ninth foot could take place.

TABLE 5.—MOISTURE RECORDS FROM TEST PLOT AT NORTH MESA.

Depth of boring, in feet, below the surface. (1)	Moisture equivalent. (2)	PERCENTAGE OF MOISTURE BY WEIGHT OF DRY SOIL.				
		Feb. 4, 1927. (3)	Feb. 10, 1927. (4)	Feb. 21, 1927. (5)	Mar. 7, 1927. (6)	Mar. 7, 1927.* (7)
1	11.6	6.8	11.2	10.9	12.5	11.6
2	11.8	6.3	10.8	11.3	9.7	10.2
3	12.9	4.2	11.1	11.1	10.3	10.3
5	11.4	4.4	4.9	12.3	12.4	10.8
7	9.8	4.0	4.2	10.8	8.3	4.1
9	9.8	4.3	4.4	4.4	7.0	4.9
11	9.2	....	....	....	7.3	4.4
13	5.2	....	....	....	3.1	3.4
15	15.1	....	....	....	10.0	15.4

* Borings taken about fifty paces from the test plot which received no moisture other than that from the rainfall.

No storms of any consequence occurred in this vicinity from 1922 until April, 1926, when about 8¾ in. fell in one storm. Subsequent storms were 1 in. in November, 1926; 3.6 in. in December, 1926; and 10.58 in. in February, 1927, the last one being observed at the test plot. The daily rainfall preceding and during the test is approximately the same as shown in Fig. 5.

TABLE 6.—QUANTITY OF WATER APPLIED TO TEST PLOT AT NORTH MESA.

Date.	Water applied, in inches.	Kind.	Remarks.
February 4.	....	....	Moisture determined from Table 5, Column (3).
" 6.	0.63	Rain water	Measured in gauge.
" 5.	4.24	Top water	Applied from a measured tank.
" 10.	....	....	Moisture determined from Table 5, Column (4).
" 12.	0.52	Rain water	Measured in gauge.
" 18.	7.99	Rain water	Measured in gauge.
" 21.	....	....	Moisture determined from Table 5, Column (5).
March 7.	1.44	Rain water	Measured in gauge.
" 7.	....	....	Moisture determined from Table 5, Columns (6) and (7).
" 7.	....	....	Samples taken from Table 5, Column (2).
....	14.82	....	Total water applied on plot.

From Column (3), Table 5, it is apparent that at the test plot the heavy rain of December, 1926, did not penetrate below the second foot and that as a

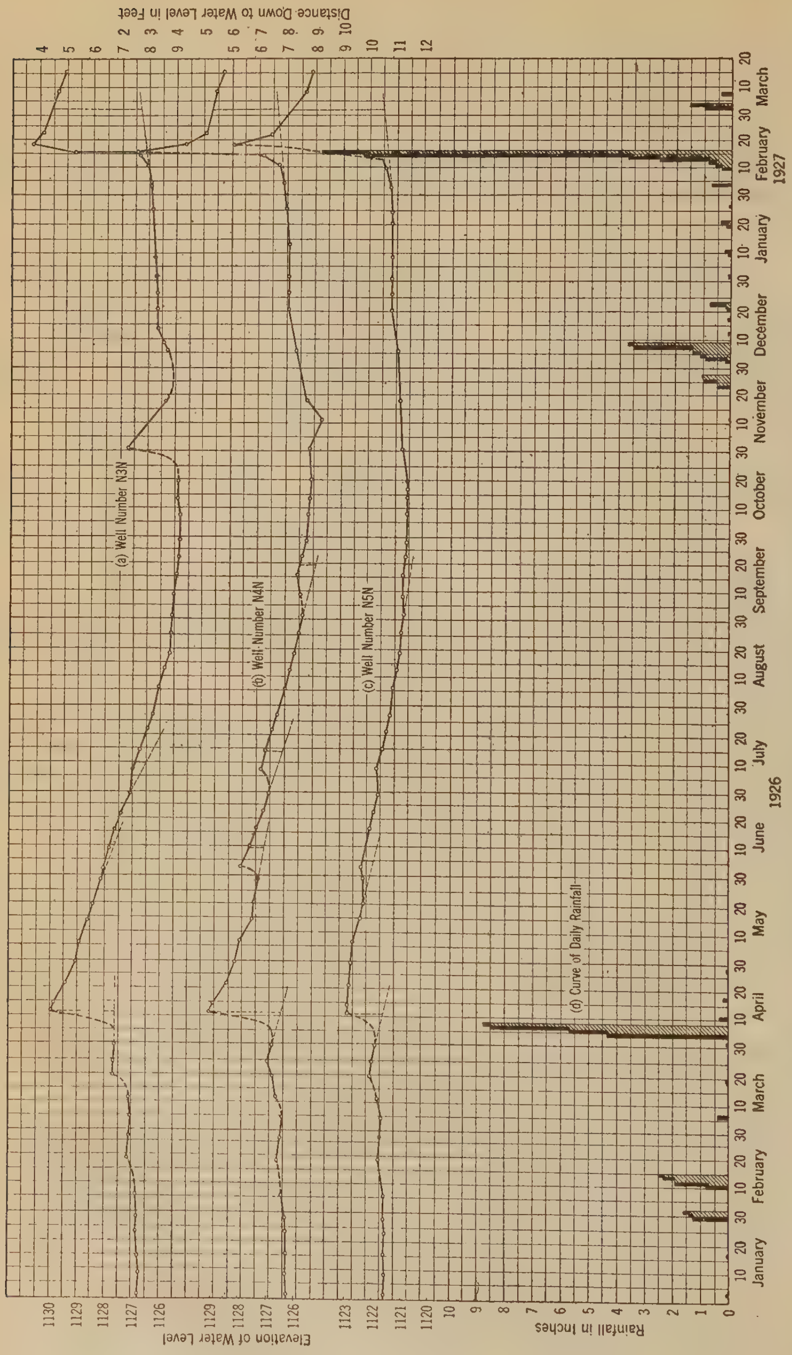

FIG. 5.—FLUCTUATIONS OF WATER LEVELS IN TEST WELLS, PAUBA VALLEY, CALIFORNIA.

result of a series of dry years the soil had dried out to the ninth foot, or lower, because, on February 4, the moisture content from the third to the ninth foot was below the wilting point. The increase in moisture after February 4 is indicated in Columns (4), (5), and (6), (Table 5). By February 21, moisture in the test plot had reached the seventh foot and by March 7, probably the fifteenth foot.

Column (7), Table (5), gives the penetration from 10.58 in. of rainfall at a point on flat ground fifty paces from the test plot. It is certain that moisture had reached the fifth foot. The excessive moisture found in the fifteenth foot may have existed prior to February 4.

A bore hole was made on February 26 in a "hog wallow" near the test plot where some water had been standing prior to that date. The log showed a dark brown sandy loam and gravelly loam to 15 ft., where a pure sand was struck. The soil was either moist or wet to 16 ft., indicating deep penetration from rainfall of 9.14 in., which occurred from February 5 to 21, 1927, due to the effect of the concentration of rain water in a depression.

Expressing the values of Columns (3) and (6), of Table 5, in surface inches of water, the writer has computed the penetration, in inches, below the ninth foot as follows:

March 7, total moisture in the top 9 ft.	18.17
February 4, total moisture in the top 9 ft.	8.66
Gain in moisture	9.51
Total water applied	14.82
Unaccounted in top 9 ft. of soil	5.31
Estimated evaporation losses, four separate wettings at 0.5 in.	2.00
Penetration below ninth foot	3.31

The foregoing results lead to the conclusion that deep penetration occurred in depressions. This is also indicated by the reaction of the water-table, underlying this mesa, to the February storm, and registered in the depth to water as shown in Table 7 (see Fig. 4).

The mesa formation is alluvial, unassorted, interpacked gravel and clay, alternating with strata of granitic gravel, probably laid down by sheet floods under a desert climate, and now under erosive action. Water-bearing gravels in five wells, which range from 200 to 600 ft. in depth, average about 30 per cent. Outcrops indicate that individual strata cover large areas, which may explain the lag and irregularity observed in the rise of the water-plane and point to temporary storage on top of clay strata.

It is reasoned that, with two or three consecutive wet years as they have been recorded in this region, deep penetration would have been general over the mesa and that an appreciable portion of the rainfall would have reached the water-table. Positive proof of this is furnished by the flow of certain springs which issue at the foot of the steep southwesterly escarpment of the

North Mesa and by the output of some artesian wells near the springs which have been flowing for several years. The location of these artesian springs and wells is marked on Fig. 4 near Well No. 14. Careful analysis of surface-water and ground-water conditions indicate that there is no extraneous, or distant, source for this water supply and that local rainfall alone is responsible therefor. There is corroborative testimony that the flow of the wells and springs varies little over a period of years, the inference being that the comparatively compact formation acts as a regulator of an otherwise erratic recharge.

TABLE 7.—Elevation of Water-Table at North Mesa in 1927.

Well No.	Date.	Distance down to water-table, in feet.	Remarks.
129	August*	154.00	......
	February	154.16	......
	March	154.40	......
	April	154.25	......
	May	154.46	......
	July	154.20	......
L8N	February 23	123.48	0.29-ft. rise (Feb. 23 to Mar. 9)
	March 9	123.19	
	March 16	123.47	0.36-ft. drop (Mar. 9 to Apr. 4)
	April 4	123.55	
	May 1	123.30	0.25-ft. rise (Apr. 4 to May 1)
131	January	63.39	1.21-ft. drop (Jan. to Mar.)
	March	64.60	
	May	63.70	1.30-ft. rise (Mar. to July)
	June	63.40	
	July	63.30	
14	March 21	38.49	1.10-ft. rise (Mar. 21 to May 7)
	April 1	38.00	
	May 7	37.39	
	June 4	37.52	0.72-ft. drop (May 7 to Sept. 24)
	July 2	37.59	
	August 8	37.86	
	September 24	38.11	

* This reading was made in August, 1926.

Measurements of flow were made on March 11 and October 26, 1927. On the latter date, the output was slightly less than in March which was thirty days after a heavy storm. In October, the springs proper flowed 0.191 sec-ft., and the artesian wells, 0.1672 sec-ft. (total flow, 0.3582 sec-ft.). Making a further allowance for evapo-transpiration losses in swamps and for wild growth in the vicinity of the springs, the total visible water supply may be estimated at 0.40 sec-ft., which is equivalent to 290 acre-ft. per year. The surface water-shed tributary to these springs and wells was computed to be 5.5 sq. miles.

The ground-water basin extends from Murrietta Creek easterly to the line marked "Granite" in Fig. 4. The direction of percolation is indicated by arrows and, in some places, crosses the surface drainage lines. The probable ground-water area tributary to the springs and wells is estimated at 7.34 sq. miles. Assuming that there is an average drainage area of 6.5 sq. miles, that the springs and wells represent the entire supply available at the bluff, and that there is no by-passed underflow, the yield is 45 acre-ft. per sq. mile, or 0.07 acre-ft. per acre.

Table 3 shows that for the 14 years from 1913 to 1927 the mean rainfall in this vicinity was 14.57 in. and that there were 5 very wet years, with an average of 22.76 in. (= 1.9 ft.). If the recharge is assumed to have taken place in these 5 years, the mean percentage of deep percolation of a wet year was $\frac{14 \times 0.07}{5 \times 1.9} = 10.3\%$ of the seasonal rainfall of 22.76 in.

Determination of Deep Penetration and Storm Run-Off by Comparison with Mountain Run-Off

The quantity and general character of the rainfall, as well as the physical characteristics that might be peculiar to an individual mountain water-shed, are expressed in the rainfall-run-off relations, and these relations can be applied with modifications to conditions on adjacent alluvial plains. A study is here presented with this end in view and illustrated for a concrete case, namely, the application of the observed rainfall-run-off relations of the San Bernardino Mountain water-shed in Southern California to the determination of deep penetration from rainfall in the San Bernardino Valley.

Reliable run-off measurements of mountain water-sheds are generally available while rainfall records, as a rule, are incomplete. The reverse is the case for valleys. Rainfall increases with altitude, in Southern California, up to about Elevation 6 000 ft. The average rainfall of a water-shed can be determined from available records, either by the construction of isohyetose lines or the use of formulas.

Effect of Different Soil Formations.—The modern alluvial valley fill which has its origin in the residual soil of a granitic range is generally more open and, for similar storm and cover conditions, should produce a larger percentage of deep penetration than the residuum. Similarly, the product of erosion of an ancient alluvium will be more porous and subject to more rapid penetration than its mother deposit. An ancient compact alluvium has a larger specific retention than a modern stream deposit and will hold larger volumes of water, subject to evapo-transpiration losses, and, therefore, will produce less available supply. This is demonstrated in Fig. 6* by comparing the run-off curves of the Santa Ana River water-shed with those of the water-sheds of Lytle, Mill, San Antonio, Devil Canyon, and Waterman Canyon Creeks.

The water-shed of Santa Ana Canyon is characterized by flats and valleys covered with an ancient alluvium, while in the water-sheds of the other streams mentioned residual soils and slides are the predominant features, and although the Santa Ana River has a higher mean seasonal rainfall, nevertheless its percentage of run-off is less than in the case of the other water-sheds.

Comparison of Physical Features Affecting Penetration and Run-Off.—On a mountain water-shed with residual soil cover, seepage water may strike bedrock at a depth of 4 to 12 ft. or more, where it will be diverted down-hill as illustrated in Fig. 7 (*a*). On this down-hill course it must run the gauntlet of absorption by the roots of trees, brush, and herbs which may have penetrated fissures to depths of 20 to 30 ft. Abstraction is, therefore, extremely active until the percolating water reaches the stream bed. Despite this continued tapping, however, seepage run-off is produced in dry years, although this may be due partly to over-year storage and partly to absorption on slides, which, owing to their unstable and porous formation, readily admit the rain water.

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* Drawn by A. D. Edmonston, Assoc. M. Am. Soc. C. E.

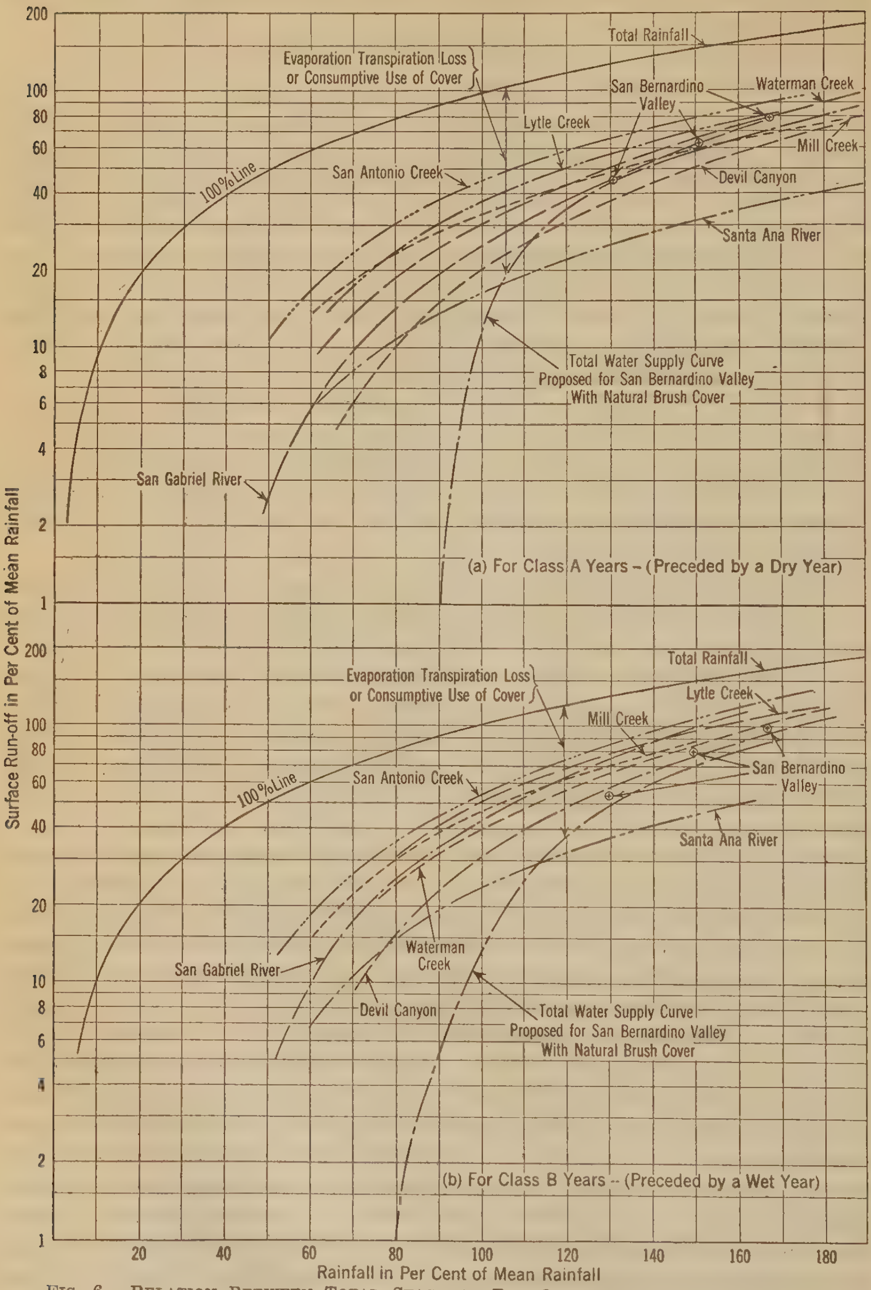
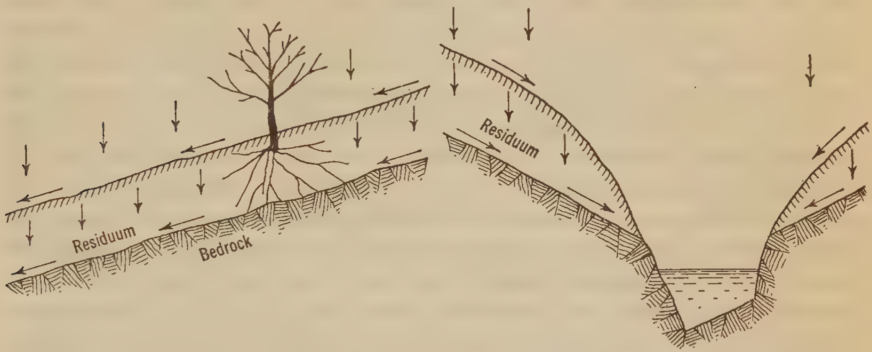


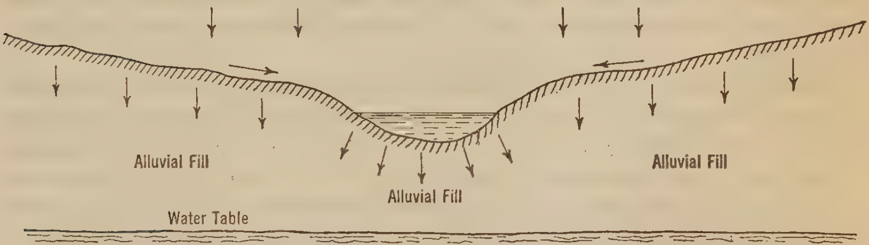
FIG. 6.—RELATION BETWEEN TOTAL SEASONAL RUN-OFF AND SEASONAL RAINFALL.

Different conditions prevail on valley floors. Bed-rock may be at depths of 20 ft. or more, and water that percolates to 10 or 15 ft. is, as a rule, permanently out of reach of plant growth and, therefore, must reach the ground-water plane. With given rainstorms of equal intensity, the valley floor with its flatter gradient, undulating surface, and heterogeneous fill, therefore, should receive a larger percentage of seepage than the mountain water-shed.



(a) DISPOSAL OF RAINFALL ON MOUNTAIN SLOPE

(b) INTERCEPTION OF SEEPAGE BY STREAMBED ON BEDROCK



(c) LOSSES BY SEEPAGE FROM STREAMBED IN ALLUVIAL FILL

Fig. 7.

A similar relation obtains in regard to seepage from stream beds. Mountain streams run on bed-rock, receiving seepage from the banks as well as surface run-off and, as a rule, they suffer no percolation losses. In valley streams the reverse is the case, there being substantial seepage losses through the sides and bottom of the stream bed (see Fig. 7 (*b*) and Fig. 7 (*c*)).

Other conditions being equal, physical conditions are, therefore, more favorable to the absorption of rainfall and the conservation of seepage in valley areas than in mountain water-sheds.

Analysis of the Rainfall-Run-Off Curve.—The run-off of a mountain water-shed can readily be segregated into surface or storm run-off and seepage or normal run-off by an analysis of the daily hydrograph and records of daily discharge. Seepage run-off is defined herein as that portion of the rain which seeps into the ground and, subsequently, is the source of the more or less perennial flow of a stream; it corresponds to deep penetration in the valley. Storm-water or surface run-off is that portion of the rainfall that reaches a stream bed on the surface during a storm and for a few days thereafter, and produces the flood flow.

Fig. 8 shows the total mountain run-off curve and its segregation into storm and seepage run-off for Devil Canyon in the Arrowhead water-shed north of San Bernardino and for which the rainfall of Squirrel Inn Station is characteristic. The water-shed covers 6.3 sq. miles; is steep, granitic, and covered with a dense growth of brush and trees. The profile shown in Fig. 9 is also characteristic of this water-shed. The total rainfall-run-off of Fig. 8 is typical of mountain water-sheds in general. The difference between the 100% or total rainfall line and the total run-off curve represents the evapo-transpiration losses, or consumptive use of the cover. It is important to note that a run-off of 2 in. occurs with a seasonal rainfall of 22 in., leaving 20 in. consumed by plant life and evaporation losses. These losses gradually increase to a maximum as the run-off curve approaches a parallelism with the rainfall line, the maximum being about 30 in. Seepage run-off, therefore, takes place during years of very deficient rainfall, when the maximum demands of plant life and evaporation are only 62.5% satisfied. Storm run-off is apparently only a minor factor at Devil Canyon water-shed. This is illustrated by Fig. 10, which is a comparison of rainfall-storm run-off curves for water-sheds in the San Gabriel and San Bernardino Ranges. For a valley floor, with its flat gradients, the storm run-off is a still smaller percentage of the total water crop unless the formation is clay.

Comparison of Relation of Intensity of Maximum Storms to Seasonal Rainfall.—The rainfall-run-off curve of a mountain water-shed, if applied directly to the valley, would give erroneous results, for the following reasons.

In Central and Southern California, storms are of the cyclonic type; they are general and visit valleys and mountains simultaneously so that the number of storms and their relative intensity is correlated. Broadly speaking, a mountain water-shed and its adjacent valley area bear the same index of wetness* for any one season. The same quantity of seasonal precipitation may be produced in the valley by a wet year and on the mountain water-shed by a dry year, but the number and intensity of storms will be quite different, to the effect that the resulting water supply will be larger for the valley. This is illustrated in Fig. 11 by a comparison of the relation of seasonal rainfall to maximum storms for the Valley of San Bernardino and the Arrowhead Mountain water-shed which is immediately to the north. Rainfall at the City of San Bernardino has been taken as representative of the valley and that of Squirrel Inn Station on the crest as representative of the mountain water-shed.

Fig. 9 shows the topographical relation of the rainfall stations in the valley and the Arrowhead water-shed. Fig. 11 shows the relation of seasonal rainfall to maximum storms. Although the points indicating individual years are irregularly located, the curves, nevertheless, depict the general character

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* The "index of wetness" is the percentage of the long-period mean. A seasonal rainfall of 20 in. would have an index of wetness of 125 for a long-period mean of 16 in.

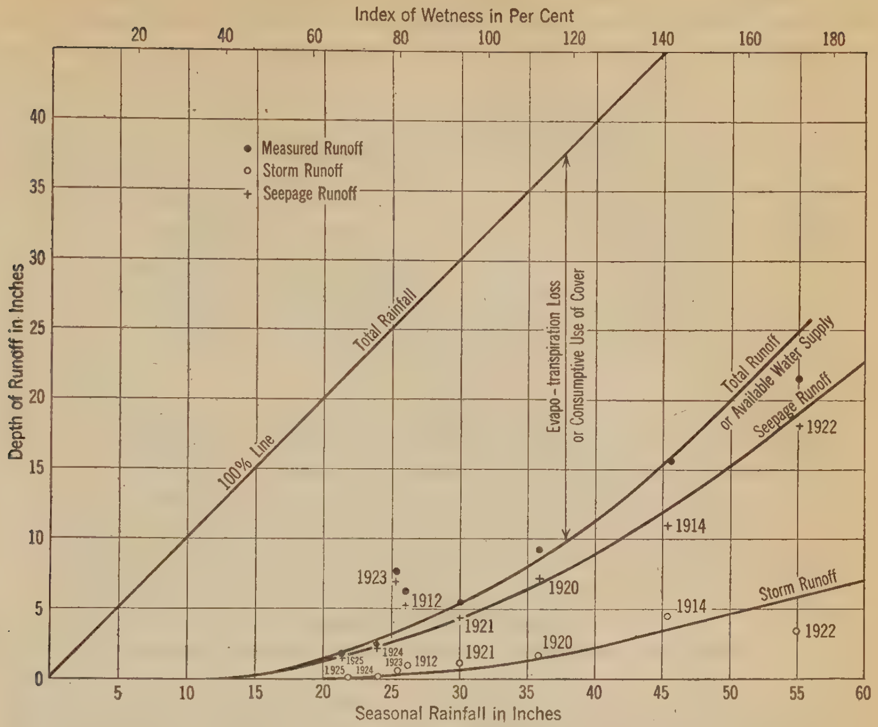

FIG. 8.—RELATION OF SEASONAL RAINFALL TO MEASURED TOTAL RUN-OFF SEEPAGE AND STORM RUN-OFF FOR DEVIL CANYON WATER-SHED.

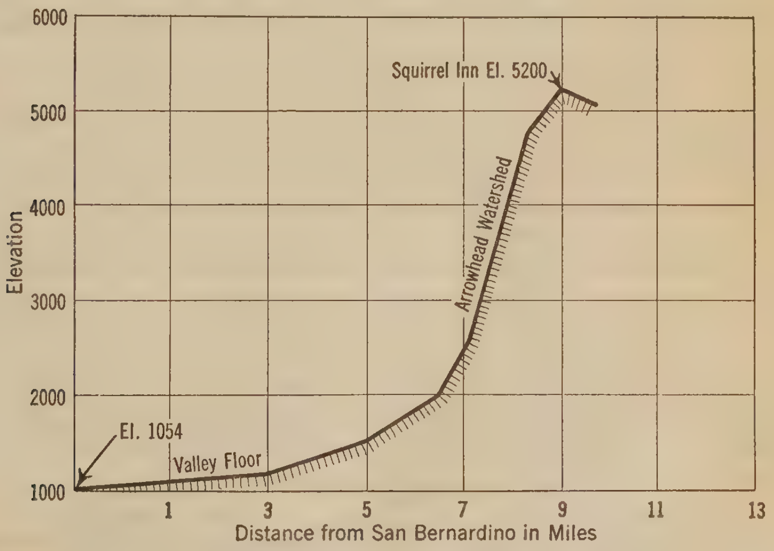

FIG. 9.—PROFILE OF ARROWHEAD WATER-SHED.

of the correlation between mountain and valley rains. For example, in order to produce a 7.5-in. storm a seasonal rainfall of 21 in. is required at San Bernardino and one of 30 in. at Squirrel Inn Station.

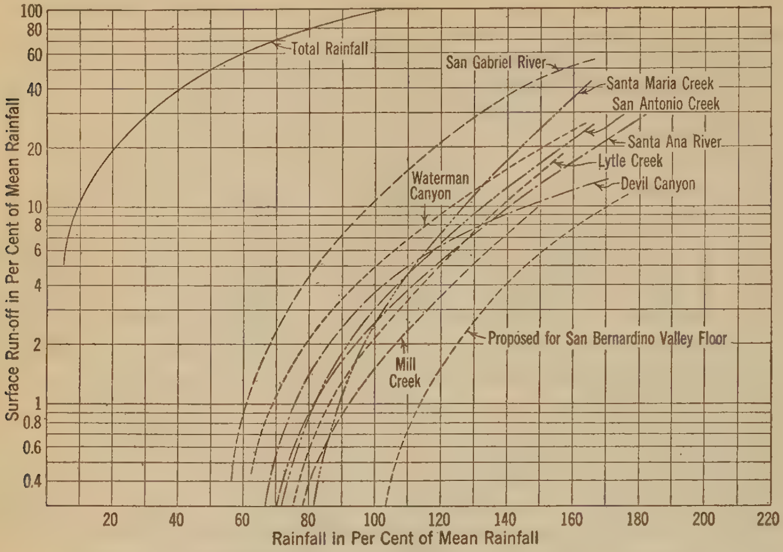

FIG. 10.—COMPARISON OF RAINFALL-STORM RUN-OFF CURVES.

Fig. 12 shows the relation of the indices of wetness to the maximum storms. It is apparent that in the mountains, as well as in the valley, heavy storms do not occur for seasonal rains below normal. In Fig. 11, the dashed line represents the "maximum storm-rainfall curve" for the Arrowhead water-shed near San Bernardino, to which Devil Canyon and Waterman Canyon belong.

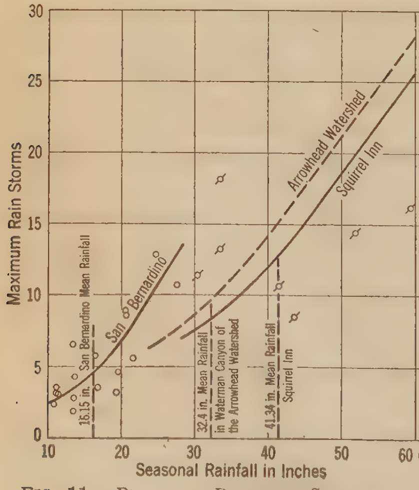

FIG. 11.—RELATION BETWEEN SEASONAL RAINFALL AND MAXIMUM STORMS.

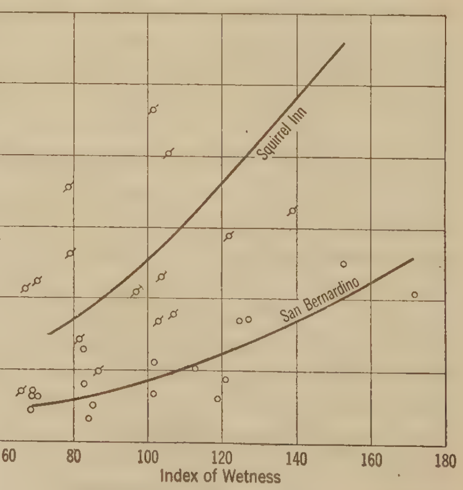

FIG. 12.—RELATION OF INDICES OF WETNESS TO MAXIMUM STORMS.

A study of Figs. 9, 11, and 12, leads to the conclusion that, for equal seasonal rainfall, the valley receives storms of much greater intensity than the mountain water-shed and produces a correspondingly larger water supply.

Determination of Valley Water Supply from Relation of Maximum Seasonal Storms of Valley and Mountain Water-Sheds.—Inasmuch as intensity of storms is a governing factor affecting penetration, as well as storm run-off, deductions from the rainfall-run-off curve of a mountain water-shed to the water supply of an adjacent valley area should take into consideration the relation of maximum storms of equal magnitude, at least as far as seasons of excessive rainfall are concerned. It is not unreasonable to assume that, for similar character of soil and cover, seasons of equal maximum storms will produce in the valley as large a water supply as in the mountains. It has been shown that conditions are more inducive to deep penetration and less inducive to storm run-off.

For example, referring to Fig. 11, a maximum storm of 10 in. corresponds to a seasonal rainfall of 24 in., at San Bernardino and of 33 in. on the Arrowhead water-shed. The total run-off curve for Devil Canyon, in the Arrowhead water-shed (see Fig. 8) gives, for a 33-in. rainfall, a run-off of 7 in. It is concluded, roughly speaking, that the water supply at San Bernardino, resulting from a 24-in. rainfall, should be equal to, if not more than, 7 in.

For the purpose of illustration, there is here presented a determination of the water supply of San Bernardino Valley in the general vicinity of the City of San Bernardino, from the known run-off of the water-sheds of Lytle Creek, Waterman Canyon, and Devil Canyon Creeks. These streams are responsible for the modern valley fill of San Bernardino. They are of granitic and schistose formation and have a dense brush and tree cover. The Valley of San Bernardino is a sloping plain and for the purpose of this illustration is assumed to have a natural brush cover. The characteristics of seasonal rainfall and the analysis of the storms of a wet season for the stations of San Bernardino, Devore, and Squirrel Inn—representing valley, foothill, and mountain water-sheds—are given in Tables 8 and 9.

The determination is made along two distinct lines: First, on the assumption that, for areas comparable in formation and cover, seasons of equal storms will produce in the valley a supply equal to, if not larger than, that in the mountains; and, second, on the basis that the relation of seasonal rainfall to the consumptive use of the cover is the same for kindred water-sheds.

Example 1.—The water supply of the valley is to be determined for three seasons in which the maximum storms were 7.5 in., 10 in., and 12.5 in., respectively.

The rainfall-run-off relations of the three mountain water-sheds—Devil Canyon, Waterman Canyon, and Lytle Creek—are shown in Fig. 6. Values for each water-shed are expressed in percentage of the long-period mean rainfall. The curve for Waterman Canyon Creek represents a fair average for the three. It belongs to the Arrowhead water-shed (see Fig. 11) and its mean seasonal rainfall is 32.4 in.

To simplify matters, the Waterman Canyon run-off curve (Fig. 6) is assumed to represent the average run-off conditions, and the Arrowhead water-

TABLE 8.—SEASONAL RAINFALL, IN INCHES.

Seasons.	San Bernardino Valley floor, Elevation 1 054.	Devore foothills, Elevation 1 990.	Squirrel Inn Mountain water-shed. Elevation 5 200.
1895– 96	8.11	......	......
1896– 97	16.74	......	......
1897– 98	8.24	......	......
1898– 99	7.49	......	......
1899–1900	8.64	......	......
1900– 01	17.36	......	......
1901– 02	11.15	......	......
1902– 03	17.42	......	......
1903– 04	9.37	......	......
1904– 05	20.78	......	......
1905– 06	19.88	......	......
1906– 07	23.17	......	......
1907– 08	15.62	......	......
1908– 09	17.44	......	......
1909– 10	15.02	......	......
1910– 11	16.40	......	......
1911– 12	13.84	......	......
1912– 13	11.08	......	......
1913– 14	21.45	......	......
1914– 15	19.59	......	43.57
1915– 16	24.72	......	77.61
1916– 17	13.79	......	41.42
1917– 18	13.33	......	33.33
1918– 19	18.62	......	28.23
1919– 20	19.28	39.95	37.20
1920– 21	16.46	35.21	46.15
1921– 22	27.75	59.99	73.58
1922– 23	11.04	30.76	41.27
1923– 24	11.34	20.32	28.42
1924– 25	10.89	26.64	34.81
1925– 26	20.40	34.16	45.23
1926– 27	20.55	37.24	43.32
Long-period mean	16.15	28.9	42.5

TABLE 9.—ANALYSIS OF STORMS OF A WET YEAR, 1926-27.

Station.	Seasonal rainfall, in inches.	RAIN FALLING IN STORMS OF:		
		Less than 2 in.	2 in. to 6 in.	6 in., or more.
San Bernardino	20.55	11.78	......	8.77*
Devore	37.24	......	3.94* 2.45 2.76	......
Total at Devore	......	12.34	9.15	15.75*
Squirrel Inn	43.32	......	2.22* 3.25 3.00 3.75 3.25	......
Total at Squirrel Inn	......	7.55	15.47	20.80*

* One storm.

shed curve of Fig. 11, to represent average relations between rainfall and maximum storms for the mountain water-sheds. The seasonal rainfall at San Bernardino and its rainfall-storm relations (as shown in Figs. 11 and 12) are representative of the valley.

TABLE 10.—RUN-OFF DETERMINATIONS, EXAMPLE 1.

Location. (1)	Storm, in inches. (2)	Seasonal rainfall, in inches. (3)	Percentage of mean rainfall. (4)	TOTAL RUN-OFF:			
				Class A Years:		Class B Years:	
				In inches. (5)	In percentage of mean rainfall. (6)	In inches. (7)	In percentage of mean rainfall. (8)
Waterman Canyon......	7.5	27.5	85	7.2	....	8.8	....
	10.0	33.0	100	10.1	....	12.7	....
	12.5	37.0	117	13.0	....	16.3	....
San Bernardino Valley........	7.5	21.0	180	7.2	45	8.8	54
	10.0	24.0	149	10.1	63	12.7	79
	12.5	27.0	174	13.0	80	16.3	100

On Fig. 6 (*b*), the points marked "San Bernandino Valley" correspond with the values in Column (8) of Table 10. These points might serve as the basis for a water supply curve of the valley for wet years.

*Comparison of Evapo-Transportation Losses.**—In general, it must be assumed that the character and volume of water supply of a country are reflected in the type of its natural vegetation. The water supply of a water-shed is expressed by the average seasonal rainfall or the long-period mean. The average water needs of the vegetation sustained by this mean should bear a more or less fixed relation to the supply.

In a semi-arid country like Southern California, with a pronounced dry and rainy season, and erratic fluctuations of rainfall—not only from day to day, or month to month, or year to year, but also over periods of years—a type of vegetation is produced that is capable of living through a series of comparatively dry years as well as of responding to a wet winter. For a valley area like that at San Bernardino, with a seasonal supply varying from 8 in. to 26 in. and an average of 16 in. (see Fig. 11), the natural vegetation produced would be chaparral, growing to heights of 3 to 4 ft., besides a variety of grasses and herbs, with a consumptive use of 15 to 18 in. At Squirrel Inn the seasonal rainfall averages about 42 in., with a minimum of 26 in. and a maximum of about 80 in. This rainfall, combined with an altitude of more than 5 000 ft., produces chaparral as high as 10 ft. and pine timber requiring for its support a water supply of 25 to 30 in. In both instances the type of vegetation would seem to bear a definite relation to the water supply.

If water-sheds have similar soil formation and are under like climatic conditions, and particularly under the régime of the same cyclonic storms, it is concluded that the relation of the consumptive use of their cover to their respective water supply is approximately the same, and that for the same season each water-shed will consume about the same percentage of its seasonal supply. The relation of seasonal rainfall to consumptive use is indicated by the rainfall-run-off curve in Fig. 6. If, then, for a series of neighboring water-sheds, both the seasonal rainfall and the total run-off—or its complement, the consumptive use—are expressed in percentage of the long-period mean rainfall, the resulting run-off curves are reduced to a common basis and should not only assume the same shape, but should also fall close together. That this is actually the case for seven water-sheds studied by the writer, is demonstrated by the curves in Fig. 6. The characteristics of these water-sheds are listed in Table 11.

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* In co-operation with A. D. Edmonston, Assoc. M. Am. Soc. C. E., and J. H. Peaslee, Esq.

TABLE 11.—CHARACTERISTICS OF WATER-SHEDS.

Name.	Area, in square miles.	ELEVATION.			Mean seasonal rainfall,* in inches.	Description.
		Minimum.	Maximum.	Mean.		
San Gabriel River...	222.0	1 000	10 080	4 400	31.0	Steep granitic; some cliffs; flats of old alluvium a minor factor; generally well forested; flashy floods.
San Antonio Creek..	16.9	2 000	10 080	6 600	43.5	Granitic; steep; absorbent detrital slopes; well wooded; flashy floods, but also relatively good summer flow.
Lytle Creek.........	47.7	2 210	10 080	5 500	32.7	South fork. steep and barren, with flashy floods; middle fork, steep and wooded; north fork (the main summer water supply), absorbent detrital slopes; wooded.
Devil Canyon........	6.1	1 750	5 250	3 600	32.7	Granitic; steep; some old alluvium flats and detrital slopes; heavy brush cover; alders and sycamores in bed.
Waterman Canyon..	4.5	1 500	5 100	3 600	32.4	Granitic; steep; heavy brush cover; alders and sycamores in bed; no flats, but good detrital slopes at the upper end.
Santa Ana Canyon..	199.0	200	11 500	6 700	37.2	Extensive flats and suspended valleys of ancient alluvium; in the upper water-shed, slopes above flats are broken quartzite and schist which is very absorbent; upper water-shed to the east shows typical desert mountain forest; slopes generally well wooded; extensive meadows.
Mill Creek..........	43.3	800	11 500	6 700	35.8	Steep; granitic; well forested; absorbent broken schist in the upper slopes.

* Estimated from available records for valley stations.

The higher of these water-sheds receives snow above Elevation 5 000 which, in wet years, is a factor that favors late run-off. Many have absorbent detrital slopes which are well wooded.

The close relation of the run-off curves of these water-sheds, both as to alignment and position, particularly those of similar characteristics, is striking and supports the contention that the known rainfall-run-off relation of a water-shed will permit of deductions as to this relation, not only of a neighboring mountain water-shed, but also of a valley floor of kindred formation and cover. In the application to a valley floor, it is to be considered that, for wet years, conditions in the valley favor larger penetration but less run-off. On the other hand, it is not likely that any water supply will become available for indices of wetness less than 70, because the rainfall of very dry years falls in light storms and is consumed. The character of the cover and its relative consumptive use as well as the over-year effect of the preceding year has an important bearing on the whole question.

Another solution for the determination of the water supply of San Bernardino Valley is outlined in Example 2.

Example 2.—Reference is again made to the run-off curves of Fig. 6, of which the Waterman Canyon curve presents a fair average for indices of wetness greater than 100. The proposed valley water supply curve, therefore, should be located near the Waterman Canyon curve. Equal weight must be given to the points marked "San Bernardino Valley", which were obtained in Example 1 on the basis of the rainfall-storm relations.

For dry years, (Fig. 6 (*a*)), the position of the curve is governed by the zero point. For ordinary dry years the rainfall at San Bernardino is about 11 in., or about 70% of the mean, with maximum storms of 2.5 to 3.5 in. On the assumption that the valley has a natural brush cover, the zero point has been estimated as follows: For Class A years, at 90% of normal, or 14.5 in.; and for Class B years, at 80% of normal, or 12.9 in. The curves have been drawn to follow the general alignment of the mountain run-off curves and marked "Proposed for San Bernandino Valley" (see Fig. 6).

Comparison of Storm Run-Off.—The relation of seasonal rainfall to storm run-off, expressed in percentage of the long-period mean, is shown in Fig. 10. Santa Maria Creek, in San Diego County, has an average seasonal rainfall of 19.4 in., and is characterized by a fringe of low granitic mountains and a comparatively flat valley of a clayey, rather impervious soil.

The data shown on Fig. 6 demonstrate that storm run-off is a minor factor, even for steep water-sheds. As indicated in Fig. 12, the valley floor will receive no storms for seasonal rainfall below normal which would produce surface run-off. The zero point of the valley storm run-off curve, therefore, has been assumed at the 100% point; otherwise, it has been drawn parallel to the general alignment of the mountain storm run-off curves. The water supply resulting from rainfall on the valley floor of San Bernardino Valley, as indicated by the proposed curves in Figs. 6 and 10, is given in Table 12. The methods presented permit of modifications which may be necessary, due to differences in the character of soil and cover between mountain water-shed and valley floor.

Determination of Rainfall Penetration from Test Wells in Pauba Valley, on Temecula River, Riverside County, California*

Quantitative determinations of deep penetration from rainfall may be made from the rise of the water level in wells. With a water-table relatively close to the surface so that intervening clay strata will not unduly interfere with percolation, the reaction on the water-table from individual storms can be traced distinctly and may serve as a basis for computations of the quantity of recharge.

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* In co-operation with B. C. Williams, Assoc. M. Am. Soc. C. E.

Observations to this end, which were made on about fifty test wells covering an area of 400 acres in Pauba Valley, and the analysis of the results, are believed to be of general interest. Incidentally, the volume of return water from irrigation was also determined for the same area.

Pauba Valley is located on the Temecula River at an elevation of about 1 100 ft. above sea level (see Fig. 4). It has a width of ¼ mile to 1 mile and a length of more than 5 miles. The average seasonal rainfall is 15.5 in., the minimum 6 in., and the maximum 24 in.

The bottom-lands on which the observations were made consist of a modern alluvium of alternating layers of fine micaceous silt and sand with a top stratum of either silt or sand mixed with humus. The adjoining mesa lands rise about 200 ft. above the valley; they have a gravelly clay loam soil and contribute to the top soil of the bottom-lands.

The following is the log of a test hole in the bottom-lands:

Distance, in feet, below surface.	Material.
0–2	Medium silt loam.
2–4	Sandy loam.
4–6	Heavy silt.
6–7	Fine sand.
7–8	Coarse sand.
8–9	Heavy silt.
9–10	Sand medium.
10–14	Coarse sand.
14	Water-table.

Ground-water stands from 12 to 20 ft. below the surface. The land is planted to alfalfa and is irrigated monthly 4 to 5 in. in depth, for 8 months per year, a record of the water applied being available.

In order to observe the penetration of bottom-lands by rainfall or return water, a large number of 2-in. test wells were driven to depths of 20 ft., perforated at the bottom. These wells are located along lines marked *G, F, H, I,* etc., in Fig. 4, and cover an area of 420 acres. Well observations were made, as a rule, once each week during the period from January, 1926, to March, 1927.

Southern California experienced a series of dry years from the summer of 1922 to April, 1926. An 8.75-in. storm in April, 1926, produced rainfall seepage. In February, 1927, a storm of abnormal magnitude (14.65 in.), occurred which also left a perfect record, while the 3.70-in. storm of December 9, 1926, was a sudden downpour of which a large part ran off.

Representative curves of well observations for 1926-27 are given in Fig. 5, which show the fluctuations of the water-plane along parts of the *N* line (Fig. 4), resulting from rainfall and irrigation. The daily rainfall (accumulative for a storm) and the approximate location of irrigation applications and of the pumping operations at Well No. 30 on the *E* line, about 6 000 ft. up stream from the *N* line, are also shown.

The effect of a storm was determined by measuring the rise in the well some time after the storm when the normal movement (drop or rise) had been re-established. The points of measurement are marked by light, dotted lines in Fig. 5. A porosity factor of 25% was allowed. This would permit some loss of water by plant life. In some instances the measurements indicated a rise due to run-off, from adjoining mesa lands. On the other hand, there was a compensating run-off away from bottom-lands.

TABLE 12.—Water Supply from Rainfall in the San Bernardino Valley.

Seasonal rainfall, in inches.	Index of wetness.	Available water supply, in inches.	Deep penetration, in inches.	Storm run-off, in inches.	Consumptive use, in inches.
Class A Years.					
12.9	80	......	......	......	12.90
14.5	90	0	......	......	14.50
16.15	100	2.0	2.00	......	14.15
17.80	110	4.0	3.89	0.11	13.80
19.40	120	5.8	5.56	0.24	13.60
21.0	130	7.25	6.85	0.40	13.75
22.6	140	8.40	7.75	0.65	14.20
24.2	150	9.50	8.60	0.90	14.70
25.8	160	10.50	9.50	1.00	15.30
27.5	170	12.0	10.40	1.60	15.50
Class B Years.					
12.9	80	....	....	......	12.90
14.50	90	0.82	0.82	......	13.68
16.15	100	2.10	2.10	......	14.05
17.80	110	4.35	4.24	0.11	13.45
19.4	120	6.30	6.06	0.24	13.10
21.0	130	8.25	7.85	0.40	12.75
22.6	140	9.70	9.05	0.65	12.90
24.2	150	11.30	10.40	0.90	12.9
25.8	160	12.9	11.90,	1.00	12.90
27.5	170	14.50	12.90	1.60	13.00

On the basis of an average rise of the water-plane for the entire area, the curve, Fig. 13 (*a*), was constructed, which shows the relation of intensity of storm to the quantity reaching the water-table. The total effect of the seasonal rainfall in 1926-27 of 23.70 in. on an area of 420 acres, was estimated at 215 acre-ft.

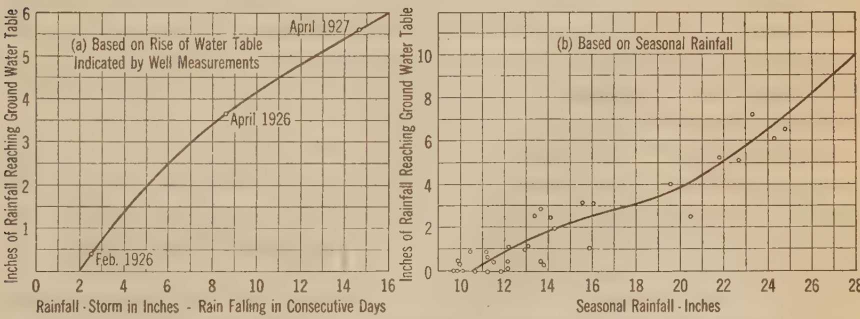

Fig. 13.—Percolation from Rainfall in Pauba Valley, California.

The relation of deep penetration to seasonal rainfall was determined by segregating the latter into storms and applying the results to the curve in Fig. 13 (*a*). By this process the curve in Fig. 13(*b*) was obtained. It is based on seasonal rainfall at Aguanga (Elevation 1 986), Elsinore (Elevation 1 300), Greenwood (Elevation 1 400), and Station C at Pauba Ranch (Elevation 1 050), and the fluctuations of Pauba Valley test wells.

The location and effect of irrigation applications are also clearly shown in Fig. 5, and were determined in like manner as that from rainfall, the conclusion being that the irrigation return waters over a large area planted to alfalfa and receiving a seasonal application of 3.5 acre-ft., were 38.4%, or 1.34 acre-ft. per acre.

Conclusions

For an alluvial valley fill of crystalline origin, the distribution and rate of penetration of moisture from rainfall over large areas are essentially non-uniform, and percolation will concentrate in numerous well-defined ducts.

Deep penetration may occur during years of deficient rainfall when the maximum consumptive use of the cover is not satisfied; particularly is this true when the formation is modern.

Rainstorms in Southern California are principally of the cyclonic type and cover the entire country. Each season is characterized by a storm of outstanding intensity that is chiefly responsible for the water supply which becomes available.

For like seasonal rainfall, the intensity of the maximum storm of a season is greater in the valley than in the mountain water-shed and so is also the resulting water supply.

For like intensity of the maximum seasonal storm, the water supply produced in the valley is approximately the same as that in the mountains (conditions of cover and soil being similar). Given the relation of maximum seasonal storms of mountains and valley, deductions may be made from the known run-off of a mountain area to the available water supply of the adjacent valley floor.

There appears to be a more or less fixed relation between the seasonal rainfall of a water-shed and the consumptive use of its cover, which remains approximately the same for kindred water-sheds of a region. Inasmuch as the consumptive use is only the complement to the run-off, the rainfall-run-off relation of a mountain water-shed will permit of definite conclusions as to the water supply from the local rainfall of its adjacent recipient valley area, if these relations are reduced to a common basis and expressed in percentage of the mean rainfall.

Storm run-off of the valley floor is a minor factor, except for compact formations under the effect of the maximum storm of a wet year. A substantial percentage of the rainfall on the valley floor becomes available, varying with the amount of precipitation, the wetness of the preceding season, and the character of soil and cover.

The valley floor in the vicinity of San Bernardino has a mean rainfall of 16.15 in., and a modern alluvial fill of crystalline origin. Assuming a brush cover, the seasonal water supply available from rainfall over a period of years is estimated to average 4.5 to 5 in., or about 250 acre-ft. per sq. mile.

DISCUSSION

Harold C. Troxell,* Assoc. M. Am. Soc. C. E. (by letter).—Seasonal rainfall cannot be divided into the four groups suggested by Mr. Sonderegger, unless another group, that of hold-over storage, is added. During wet years a large percentage of the rainfall might remain in storage to be discharged during the following season.

Just northwest of San Bernardino, Calif., and tributary to Cajon and Lytle Creeks is Lone Pine Creek, for which records are available since 1919. In contrast to the average Southern California stream, Lone Pine Creek passes over large sand and gravel areas and its canyon side walls are gently sloping and covered with sage-brush. The gauging station is located on an outcropping of bed-rock at the extreme lower end of the canyon. There is every reason to believe that there is practically no underflow by this station. The drainage area above the gauging station is 16.7 sq. miles.

In 1921-22 this area was visited by an unusually heavy rainfall. At a rain gauge near the mouth of the canyon 43.23 in. were registered for the season and the run-off from Lone Pine Creek was 3 810 acre-ft. Although this is the surface run-off, it does not represent the run-off from the 43 in. of rainfall. A large proportion of this rain was stored in the canyon and did not drain away for some time afterward.

During the next season of 1922-23 at least 1 440 acre-ft. of the 2 090 acre-ft. run-off for that year was from rain placed in storage during 1921-22. In 1923-24, 530 acre-ft.; in 1924-25, 195 acre-ft.; and in 1925-26, 15 acre-ft. were drawn from rainfall placed in storage during 1921-22. From this it is seen that the seasonal rainfall of 43.23 in. during 1921-22 produced, not 3 810 acre-ft., but at least 5 800 acre-ft. This figure is more likely to be an under-estimate than an over-estimate, because this storage has had to help supply the transpiration and evaporation losses for at least four years.

In Fig. 6 Mr. Sonderegger has divided the seasons into two groups—those that follow wet years and those that follow dry years. Those that follow wet years are plotted so as to show more run-off than those that follow the dry years.

The accuracy of this graph is questionable, because the entire seasonal rainfall during wet years is plotted against the seasonal run-off for that year, and not against the total run-off due to that rainfall. Likewise, during the following dry years, the run-off is plotted against a seasonal rainfall that only delivered a part of that run-off.

Furthermore, for years following wet years, the transpiration and evaporation losses are shown to be less than for those following dry years. The transpiration and evaporation losses for any season are governed primarily by the rainfall of that season. However, during years following wet years these losses would undoubtedly be greater, instead of less, due to the available storage. Ordinarily, wet years will have greater losses than dry ones irrespective of whether they follow wet or dry years.

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For example, Fig. 6 shows that for years of mean rainfall Waterman Canyon Creek will have a transpiration and evaporation loss of 70% following a dry year and only 60% following a wet one. The transpiration and evaporation losses for the year following the wet year should be greater, due to the hold-over storage available to the plant life, and should be less for the year following dry years on account of lack of this storage.

A true division of the seasonal rainfall into transpiration, evaporation, storm run-off, and seepage run-off (penetration) can only be made when the quantity placed in storage during wet years is added to the seepage run-off for that year, and that drained from storage in the following years is subtracted from the seepage run-off for those years. Only one curve will be needed under these conditions. This curve will show the transpiration and evaporation losses irrespective of the character of the preceding years.

Curves, such as Fig. 8, showing a comparison between seasonal rainfall and storm run-off, should be used with a great deal of caution. Storm run-off depends not on seasonal rainfall, or even storm periods, but on the instantaneous rate of the rainfall and the condition of the canyon storage. Every acre of the canyon has a certain uniform rate by which it absorbs the rainfall if storage is available. If the rate for a given area is 1 in. per hour, then any storm that does not exceed that rate will be completely absorbed. If, for example, 1 in. should fall in ½ hour then ½ in. will run off. If this inch should fall in 15 min., then there would be ¾-in. run-off. Therefore, if a rainfall of 1 in. should occur within the hour, it would be necessary to have the instantaneous rates in order to determine the storm run-off for that rain. On the Santa Ana River the season of 1922 was a good example of this. While the first major storm of that year was as severe as those of April, 1926, and February, 1927, it did not cause any extremely high water.

Another item just as important to storm run-off as the intensity of rainfall is the condition of the canyon storage. If the voids in the canyon walls and gravels of the stream beds are completely surcharged, then the rain will run off as storm run-off irrespective of the rate of rainfall. For these reasons any comparison of seasonal rainfall and storm run-off should be avoided.

Mr. Sonderegger states that storms in mountain and valley areas of equal size will deliver equal supply, and this is undoubtedly true. Then, by drawing a curve (Fig. 11), showing the relation of the maximum storms to the seasonal rainfall, he assumes that like maximum storms will produce like seasonal run-off. The logic of this is doubtful because the seasonal run-off (or supply) depends, not so much on the maximum storm, as on the instantaneous rate of rainfall, the distribution of the storms, and the condition of the storage at the beginning of the season.

Table 13 shows the run-off from Waterman Canyon and Strawberry Creeks for the period of record, together with the seasonal rainfall and the storm periods of more than 4 in. at Squirrel Inn.

During 1925-26 and 1926-27 the seasonal rainfall at Squirrel Inn was almost the same for both years, as were also the maximum storms. On Strawberry Creek the season of 1926-27 produced a run-off almost 50% greater than that of

1925-26. This was due to several reasons, one being that the latter season was preceded by several dry years and had very little in storage at the beginning of the season. The first storm of the season, a rainfall of 4.97 in. during the early part of October, had practically no effect on the surface run-off. It was apparently used to make up the soil-moisture deficiency and storage. From then until April, the discharge of the creek varied little, except for a short period in February following a storm of 6.90 in. The maximum storm of the season was a 19.76-in. rainfall in April. The storm left the storage in the canyon fairly well filled, insuring a normal run-off for the summer months.

TABLE 13.—COMPARISON OF RAINFALL AND RUN-OFF RECORDS, WATERMAN CANYON AND STRAWBERRY CREEKS.

Season.	Run-Off.		Rainfall (Squirrel Inn).	
	Waterman Creek, in acre-feet.	Strawberry Creek, in acre-feet.	Seasonal, in inches.	Storm periods of 4 inches, or more, in inches.
1920-21	1 840	3 170	46.15	9.01, 8.40, 7.04
1921-22	6 420	11 000	73.58	23.79, 8.68, 6.91, 6.26, 5.71
1922-23	2 230	3 680	44.27	7.73, 5.01
1923-24	784	1 600	28.42	4.06
1924-25	555	1 430	34.81	4.00
1925-26	1 160	2 520	45.23	19.76, 6.90
1926-27	2 040	3 720	43.32	20.30
1927-28	585	1 300	16.39	5.20

At the opening of the 1926-27 season, while the storage was low, it was hardly as deficient as in 1925-26. Several storms of about 2 in. in November, followed by storms at intervals of about two weeks, kept the daily discharge about normal. In the middle of February the maximum storm of 20.30 in. occurred. This was immediately followed by several storms of more than 3 in., in March. It was these March storms that caused the run-off for 1926-27 to be greater than the run-off for 1925-26. After the heavy storm of February had filled the available storage in the canyon, the storms of March were forced to run off. Had the April storm of the 1925-26 season been followed by storms such as fell during October and February of that season, the run-off for 1925-26 season might have equalled that of 1926-27.

In 1920-21 the seasonal rainfall was greater than in both 1925-26 and 1926-27 (46.15 in. at Squirrel Inn). The season opened with a storm of 4.00 in. in the latter part of October. From then, until the end of May, followed a series of storms, the greatest of which was a 9.01-in. rain in January, a 8.40-in. rain in March, and one of 7.04 in. in May. All these storms showed a large run-off, due to the fact that the canyon storage was completely charged during most of the rainy season. Unlike both 1925-26 and 1926-27 the 1920-21 season did not have a major storm; yet the run-off from Strawberry Creek was only about 15% less than in 1926-27 and 35% greater than in 1925-26, each of which had maximum storms of about 20 in.

In 1921-22, with a maximum storm of 23.79 in. (only 3.4 in. greater than that of 1926-27), the seasonal run-off was more than three times greater.

The heavy storm of December was followed by minor storms at frequent intervals. In other words, it is not so much the size of the maximum storms that effects the seasonal run-off, but the order in which the maximum and minor storms occur. Practice in Southern California has taught hydrologists that, while a storm of 3 to 5 in. in October or November might not affect the run-off, storms of 2 to 3 in. following major storms might give considerable run-off.

In order to determine the deep penetration on the valley floor, the only comparison that can be made with the mountain area, is to assume that the evaporation and transpiration losses per unit of area are equal. It seems logical that a natural brush cover in the San Bernardino Valley should have the same transpiration and evaporation losses as the mountain area for like cover.

By drawing curves similar to those of Mr. Sonderegger, showing the transpiration and evaporation losses, and seasonal run-off corrected for storage, for Strawberry, Waterman Canyon and Lone Pine Creeks, the following conclusion might be drawn as to the deep penetration on the valley floor. If the growth was as extensive as that in Lone Pine Canyon where hold-over storage exists, then for a mean seasonal rainfall of 16.15 in. the deep penetration would be about 0.2 in. If the growth were more comparable to that found in Strawberry Creek, then for a mean seasonal rainfall the penetration would be about 0.5 in. On the other hand, there might not be any deep penetration for a season of 16.15 in., if the valley floor was assumed to have a natural brush cover, because, under these conditions, the valley would be in its native state, having large moist areas acquired during wet years. The evaporation and transpiration from such areas would greatly exceed that of the mountain areas.

HARRY F. BLANEY,* ASSOC. M. AM. SOC. C. E. (by letter).—The increasing demand for water for irrigation and domestic purposes in the semi-arid areas of the Southwest has created a need for more definite information as to what proportion of rain falling on the valley floors penetrates to the ground-water supply. Mr. Sonderegger's paper is a valuable contribution to the study of this complex subject.

Reference is made to some estimates, by the writer, of winter evapo-transpiration losses and soil moisture deficiency at the beginning of the rainy season. Part of these are conclusions drawn by the author based on the writer's estimates. It should be definitely understood that these are preliminary and may be modified by subsequent information. It was necessary at that instance, to make an estimate of the contribution of the rain falling on the valley floor to the ground-water supply, for the purpose of completing the hydraulic accounting in the Santa Ana River report.†

The following is a brief description of the method used. Rainfall penetration on the valley floor below the root zone was estimated indirectly by assigning values to evaporation, transpiration, soil moisture deficiency, and surface run-off as deduced from one season's work in the Santa Ana area, and by corroborating data secured in San Diego County and San Fernando Valley. These were analyzed and deducted from each rain storm, and the remainder of the rain was considered to penetrate below the root zone and eventually reach the ground-water. Assumptions are for average soil conditions, and no attempt was made to show the effect of soil texture on the depth of penetration. Values for the factors were assigned as follows.

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* Irrig. Engr., U. S. Dept. of Agriculture, Los Angeles, Calif.

† "Santa Ana River Investigations," *Bulletin 19*, State of California, Dept. of Public Works, Div. of Eng. and Irrig., 1929.

Evaporation.—Temperature, wind velocity, humidity, type of soil, kind of vegetation, transpiration, altitude, interception of rain by vegetation, period between storms, etc., are some of the many factors that affect the evaporation loss after a rain storm.

From observations it was found that the evaporation loss from a bare soil in the winter months very nearly equals the loss from a free water surface until the time the soil drains to field capacity. After that, the surface evaporation loss was readily determined by soil sampling. The best data available indicated that the average evaporation loss from the top soil is about ½ acre-in. per acre after each rain storm.

Transpiration.—After careful consideration of unpublished data from co-operative irrigation investigations* in Northern San Diego County, covering two seasons, the total average transpiration for all active growing vegetation was taken as 6 acre-in. per acre during the six winter months, or 1 acre-in. per month. The rate of transpiration depends upon the crop. Other factors, such as temperature, air movement, humidity, available soil moisture, etc., also influence the rate of transpiration.

Investigations have shown that bare lands, vineyards, and deciduous orchards that are clean cultivated, have no material transpiration loss during the winter period.

Soil Moisture Deficiency.—In calculations of rainfall penetration, the deficiency of soil moisture below field capacity at the end of the summer season must be reckoned in analyzing the following rainy period. There will be no appreciable downward penetration until all the soil within the root zone has been filled to field capacity. The initial moisture content of the soil at the beginning of the rainy season will vary with the kind of crop, depth of root zone, type of soil, amount of irrigation, depth of water-table, etc.

After considering many observations made in Southern California, the values given in Table 14 were taken for the deficiency of soil moisture at the beginning of the rainy season, for average soil under various conditions.

Run-Off.—Hydrographers of the Santa Ana investigation determined the run-off rate per square mile on certain known areas. These rates were used also on undetermined areas.

Calculations of Rainfall Penetration.—In general, it was found that every season must be studied separately in order to arrive at the penetration for that season. The same seasonal rainfall may give entirely different penetration, due to varying intensity and distribution of storms. For computing the amount of rainfall penetrating below the root zone during the season,

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* "Irrigation Water Requirement Studies in San Diego County, Calif.", by S. H. Beckett, Harry F. Blaney, and C. A. Taylor, Univ. of California and U. S. Dept. of Agriculture.

1926-27, the values in Table 14 were used. Known rainfall records as near as possible, in total seasonal amount to the required average for an area, were used. An example of detail calculations for rainfall observed at the U. S. Weather Bureau Station at San Bernardino, Calif., is given in Table 15. The rainfall for the season, 1926-27, was 20.55 in. Table 16 contains a summary of estimates obtained for varying rainfall, by interpolating between calculated quantities.

TABLE 14.—DEFICIENCY OF SOIL MOISTURE.

Description of land.	Deficiency, in acre-inches per acre.
Bare land	1
Irrigated crops (except deciduous)	3
Grain, weeds, and grass (non-irrigated)	5
Deciduous trees and grapes	7
Native brush	7

The State Division of Water Rights of California recognizes the importance of considering rainfall penetration in water supply studies and gives some interesting data in its recent report of the San Gabriel investigation.*

TABLE 15.—EXAMPLE OF DETAIL CALCULATION OF RAINFALL PENETRATION FOR IRRIGATED CITRUS LAND, IN ACRE-INCHES PER ACRE.

Storm period.	Deficiency of soil moisture.	Rainfall.	CONSUMPTIVE USE.*		Run-off.	Penetration below root zone.
			Transpiration.	Evaporation.		
1926-27:						
November 12-13	3.0	0.15	0.5	0.2	....	....
November 24-27	3.5	2.03	0.4	0.5	....	....
December 3- 9	2.4	1.68	0.1	0.5	....	....
December 12-13	1.3	0.10	0.3	0.1	....	....
December 18-23	1.6	1.30	0.7	0.5	....	....
January 10-12	1.5	0.41	0.4	0.4	....	....
January 20-23	1.9	0.95	0.4	0.5	....	....
February 4- 5	1.8	0.93	0.4	0.5	....	....
February 11-18	1.8	8.77	0.2	0.5	0.8	5.5
February 22-24	0.0	0.13	0.3	0.1	....	....
March 3- 5	0.3	1.15	0.2	0.5	....	0.2
March 9-10	0.0	0.28	0.7	0.3	....	....
March 28-30	0.7	1.23	0.4	0.5	....	....
April 10-12	0.4	0.94	0.6	0.5	....	....
May 1	0.6	....	....	....	....	....
Total penetration						5.7

* Transpiration and evaporation after the initial data are computed for period from second date on line to second date on line below.

At the request of the State Engineer of California a co-operative investigation is now (1929) being conducted by the Division of Agricultural Engineering, U. S. Department of Agriculture, to determine the penetration and storage of rain falling upon the valley floors of the Santa Ana River area in Southern California. This study will cover a four-year period.

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* "San Gabriel Investigation", by Harold Conkling, *Bulletin* 7, State of California, 1929.

In connection with this investigation a large number of rainfall penetration stations have been established on predominating soil types throughout the area. Measurements are being made of rainfall, evaporation, transpiration, surface run-off, and deep percolation. Soil samples are taken with standard soil tubes to depths of 12 to 18 ft., which is well below the root zone of most crop and native plants. In this way the storage of rain water in the soil and the downward percolation are determined. Ten of these stations are especially equipped for intensive work.

TABLE 16.—Estimated Rainfall Penetration Below Root Zone, in Acre-Inches per Acre, for Season 1926-1927, Santa Ana River Area.

Seasonal rainfall, in inches.	Irrigated lands (excluding deciduous and vineyards).	Deciduous and vineyards.	Grain and grass land.	Brush land.	Bare land.
9	0.0	0.0	0.0	0.0	2.5
12	0.0	0.0	0.0	0.0	5.5
15	1.3	2.2	0.0	0.0	8.4
18	3.9	5.0	2.1	0.0	11.3
21	6.4	7.7	4.5	2.2	14.2
24	8.9	10.5	6.9	4.7	....
27	11.5	13.3	9.4	7.2	....
30	14.0	16.0	11.8	9.7	....

Shafts and tunnels have been dug at several locations along the foothill area where the ground is too rocky for soil sampling. The downward percolation of rain water is being measured at the tunnels. At one shaft near San Bernardino artificial rain has been applied during the past two seasons and the downward movement of water was observed at tunnels 12 and 20 ft. below the ground surface.

Since a large portion of the rainfall absorbed by the soil is lost by evaporation and transpiration, field plots are being intensively sampled throughout the year to determine the consumptive use of water by various crop plants and native vegetation.

Unfortunately, the rainfall in the Santa Ana River area has been below normal during 1927-28 and 1928-29 so that no final conclusions have been reached as to the amount of rainfall penetration.

Mr. Sonderegger is to be complimented on his excellent analysis of a method to determine rainfall penetration on the valley floors by comparison with mountain run-off. However, the writer is of the opinion that usually the yield of water in mountain areas is essentially different from that resulting from rainfall penetration on valley floors.

There is an initial deficiency of soil moisture within the root zone, at the beginning of each rainy season. This is a variable factor dependent on plant life, depths of root activity, soil type, and depth of soil. The soil on the valley floors is generally of such a depth that the root activity is a maximum under native conditions, while in the mountain areas, bed-rock usually is near the surface, and in all probability there is root activity throughout the entire soil cover, the depth being limited only by bed-rock.

Rainfall penetrating below the root zone on the valley floors is at once out of reach of the overlying cover, except in areas of high water-table. In the mountains where the soil is shallow the condition is different. There the water is continually within reach of plant roots until it escapes into the stream channels or deep alluvium. It is only when the rate of drainage toward and along the stream channels is greater than the rate of evaporation and plant transpiration that there is any material yield of water from a mountain area.

Another modifying factor is the steepness of the slopes of a mountain water-shed. There is a far greater actual area of plant cover than the projected area used in determining the quantity of water resulting from a given depth of rainfall in inches. Conditions then are different in mountain areas than on valley floors, even for native brush cover; but further than this, the major portion of the valley floors is cultivated and much of it is irrigated. The depth and character of root activity of cultivated crops are widely different from that of native brush cover.

C. W. Sopp,* Assoc. M. Am. Soc. C. E. (by letter).—The author is to be commended for his outline of methods for computing a source of water supply of great importance to communities dependent on ground-water developments.

The writer believes that, where the water-table is close to the surface, the method based on the rise of water levels is the most satisfactory if used with porosity data obtained from many well logs. This method may be applied also where the water-plane is at considerable depth, because the moisture in the alluvium between the root zone and the water-table is always practically up to field capacity.

This statement is based upon an analysis of nineteen samples of sands taken from a shaft dug in recent alluvium near Pasadena, Calif. These samples were taken from depths of from 75 to 170 ft. at the end of a season in which the total precipitation was only 13.73 in. This condition permits rapid movement of moisture, penetrating below the root zone, to the water-table, and the resulting rise of water levels, even at depths of 150 ft., is noticeable within a short time. A study of the initial rise of water levels in eight wells, observing two heavy storms for each well, showed an average rate of movement downward of 10 ft. per day. This is under previously saturated conditions of top soil.

The method of comparison with run-off from adjacent mountains is the easiest to apply, but is liable to wide differences, depending on the judgment of the engineer. The extension of the measured mountain run-off data to valleys which inherently differ in slope, soil, cover, and culture, is difficult. For example, Fig. 6, using average rainfall, shows a variation in run-off of 30% for similar mountain drainage areas.

The rainfall-run-off relation illustrated by Fig. 8, agrees with a law the writer has formulated for Southern California streams; the percentage run-off is equal to the seasonal rainfall, in inches, less ten, up to an annual precipitation of 60 in. As an illustration, with a seasonal rainfall of 60 in., the run-off, and, consequently, the evapo-transpiration losses, is 50%, or 30 in. The maximum consumptive use of forest cover on Southern California water-sheds is about 30 in.

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* Asst. Engr., Pasadena Water Dept., Pasadena, Calif.

There is an additional method which the author has not mentioned, and that is, to analyze the rainfall by storms and to consider that all precipitation—after deducting run-off, evapo-transpiration losses, and moisture required to bring the soil in the root zone up to field capacity—moves downward and ultimately reaches the ground-water. This method gives conservative quantitative results. The principal criticism is that, due to collection of rainfall in depressions, some ground-water replenishment will occur even with small precipitation.

W. P. Rowe,* Assoc. M. Am. Soc. C. E. (by letter).—The question of consumptive uses of various types of vegetation is one that is being studied by a great many investigators. These investigations, as a rule, are being made in connection with studies not directly allied with engineering but which, when finally assembled and correlated, will prove of great value to the engineer dealing with water supply.

The author has spent considerable effort in determining the consumptive use of the vegetation on various water-sheds in Southern California. Of all the water-sheds given in Fig. 6 and Table 11, Devil Canyon and Waterman Canyon are the only ones which have a brush cover similar to the valley floor area, the others, reaching to higher elevations, have considerable areas of pine and oak timber. The author states that "the Valley of San Bernardino is a sloping plain and for the purpose of this illustration is assumed to have a natural brush cover." This classification eliminates from the discussion all the water-sheds except Devil Canyon and Waterman Canyon.

Transpiration losses from these two water-sheds occur from two distinct types of vegetation. The canyon bottom with its concentration of escaping seepage water in the form of underflow and surface streams supports a water-loving growth in which alders predominate. The area above the canyon bottom supports a vegetation that is Nature's solution to the problem of a water-shed cover that will take water when it can get it and live through severe droughts. This is the true chaparral or dwarf forest. It will coppice readily after being burnt over and in a few years the cover is as thick as it was before the fire. Without this faculty of restoration, it would have been discarded by Nature thousands of years ago.

Any one familiar with the summer streams in the low altitude of Southern California, in which class Devil Canyon and Waterman Canyon belong, knows that there is a great variation between the quantity flowing in the morning and in the evening. This variation is caused almost entirely by the fluctuation in transpiration of the canyon-bottom growths, the mountain-side transpiration fluctuations being "ironed out" by the time and distance elements of percolation. In Devil Canyon the floor will fluctuate as much as 1.0 sec-ft., the morning flow being 1.5 sec-ft. and the evening flow, 0.5 sec. ft. This is not a measure of the total transpiration losses from the entire stream-bed growth because the time for the surface water in the stream to pass through this growth has a tendency to overlap the fluctuations. The writer has found that alders on the desert side of the San Bernardino Mountain have a consumptive use of 8 acre-ft. per acre per annum, which is probably greater than the losses from alders under the cooler climate as given by the author for the Arrowhead water-shed.

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The writer fails to check the author's rainfall data, as shown on Fig. 8. The elevation of the gauging station on Devil Canyon is about 1 800 ft. above sea level. The crest of the Devil Canyon water-shed meanders for a distance of about 4½ miles at an average elevation of 5 000 ft. The air-line distance across the canyon between the 5 000-ft. contours on either side is about 2½ miles. The water-shed is a typical fan-shaped one with its narrowest part at the gauging station. In Table 8, the rainfall for Squirrel Inn, at the summit of the mountain, but easterly of Devil Canyon, is given as 73.58 in. for 1921-22 and 44.27 in. for 1922-23. The rainfall for Devore at about the same elevation as the gauging station, but westerly thereof, is given as 59.99 in. for 1921-22 and 30.76 in. for 1922-23. The author omits the Arrowhead Springs rainfall record. This station is east of the Devil Canyon gauging station and had a rainfall of 39.49 in. in 1921-22 and 19.58 in. in 1922-23. It is about the same elevation as the Devore rainfall station and the Devil Canyon gauging station. The average of the Arrowhead Springs and Devore records would give the approximate rainfall at the lowest and narrowest end of the Devil Canyon water-shed. This would make the 1921-22 and the 1922-23 rainfall at the Devil Canyon gauging station 61.66 in., and 34.72 in., respectively. If these are averaged with the Squirrel Inn record to obtain the average rainfall for the Devil Canyon water-shed, the result is 61.66 in. for 1921-22 and 34.72 in. for 1922-23. This is probably conservative as the width of the water-shed increases as it approaches the crest where the rainfall is greatest.

When these figures of average rainfall for the Devil Canyon water-shed are compared with the total run-off, the total consumptive use of the water-shed vegetation is seen to be 40 in. for the wet season of 1921-22 and 27 in. for the dry season following. There was probably some hold-over seepage water from 1921-22 which was available for plant life in 1922-23, so that the average consumptive use of 33 in. for the two seasons would be representative of what the vegetation could consume. The brush near the tops of the ridges will transpire less water than that farther down the slopes because the seepage water has a tendency to escape down hill where it becomes available for growth on the lower areas after the upper area is deficient in moisture. The brush on the northern exposures, being in the shade a longer period of time, will require less moisture than that on the hot southern exposures. It will be seen from this that the average consumptive use of the cover for the entire water-shed above the canyon bottoms is made up of brush having subnormal and abnormal transpiration losses. The maximum possible consumptive use of the brush will be more than 3.0 acre-ft.

The outwash slopes of Devil Canyon and Waterman Canyon are composed of sands, gravels, and boulders with a top soil supporting a heavy growth of chaparral, chamise predominating. This growth is neither as large nor as luxuriant as that on the northern exposures within the two water-sheds, as the southerly exposures of the outwash slopes offer maximum conditions for transpiration losses. The writer was instrumental in having installed a set of overhead sprinklers on a test area at the mouth of Devil Canyon by means of which the moisture available for plant growth could be augmented by artificial rains. Experiments are now (1929) being conducted by the Department of Agriculture, under the direction of Henry F. Blaney, Assoc. M. Am. Soc. C. E., to determine the consumptive use of this brush.

The roots of the native chaparral extend to a depth of 18 ft. Water was applied to the test plot through the sprinklers during the night hours in the spring when the air was cool and there was no wind. The quantity falling on the plot was determined by means of rain gauges scattered over the area. An application of water at the rate of 1 in. per hour for 4 hours produced no run-off. The author has pointed out that, when comparing mountain growth chamisal with outwash slope chamisal, it must be remembered that in the mountain growths the root zone is limited by the depth to bed-rock, whereas in the outwash slopes the root zone is limited by the depth of rainfall penetration over an average of several years. The deep penetration of rainfall during a wet year will, no doubt, stimulate the root growth and encourage it to extend deeper, but the dry years with a lack of deep penetration will cause the deeper roots to become dormant. Any curtailment of chaparral growth on the porous outwash slopes will result in an additional quantity available for deep penetration to the underground water plane. After the deep-rooted native brush is removed and the area cultivated, but later abandoned, the native annuals soon overrun the area. These annuals have a depth of root zone of about 8 ft. which, when compared with the deep-rooted chaparral, should result in a lessened consumptive use by the weeds and a consequent greater supply to the underground water. In a region where at least 95% of the water supply is obtained from ground-water sources, this item becomes of great importance.

Fig. 14 shows the effect of fire on the chaparral cover in the mountain areas. Devil Canyon, with an area of 6.16 sq. miles above the gauging station, adjoins Waterman Canyon with an area of 4.55 sq. miles. These two canyons have identical exposures, and altitudes, are typically fan-shaped and probably have equal rainfall. One of the main traveled roads to the mountain resort area in the San Bernardino Mountains traverses the length of Waterman Canyon. There have been frequent fires in the Waterman Canyon area, while Devil Canyon is more isolated and, prior to 1924, had had no serious fires for at least thirty years. In 1911, the Waterman Canyon area was almost entirely burned over. The stream-bed growth, however, generally escapes these fires, either because of its position in the bottom of the canyon, or, because of a higher moisture content in the vegetation, or perhaps a combination of both. The cover in the Devil Canyon water-shed escaped burning by the fire of 1911.

Stream-flow observations were made on both canyons for a few years following the Waterman Canyon fire. A comparison between the stream flow of Devil Canyon and Waterman Canyon is shown in Fig. 14. The low flows in winter are omitted from the diagram to avoid confusion with summer flows which are important. It will be noticed that the flow from Waterman Canyon exceeded that from Devil Canyon throughout the year. The wet season of 1913-14, three years after the fire, produced 515 acre-ft. per sq. mile more run-off than Devil Canyon, the total for Waterman Canyon being 1 322 acre-ft. per sq. mile and for Devil Canyon 807 acre-ft. per sq. mile.

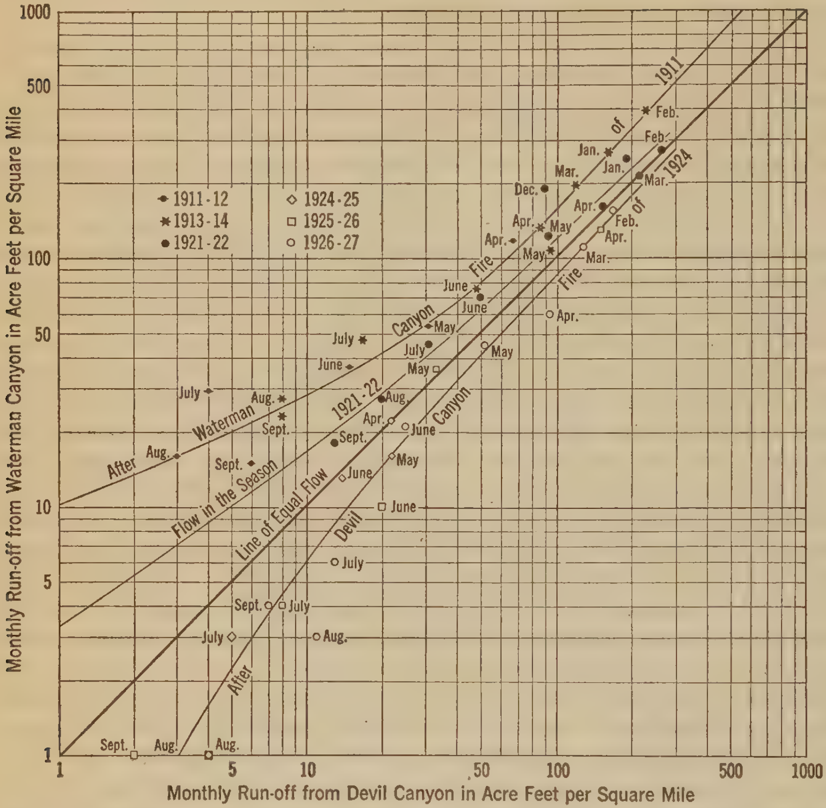

FIG. 14.—EFFECT OF FIRE ON CHAPARRAL COVER IN MOUNTAIN AREAS IN CALIFORNIA.

Inasmuch as the summer flow of Waterman Canyon was sufficient to supply the needs of canyon-bottom vegetation and yielded a surplus over the Devil Canyon flow, this excess of 515 acre-ft. per sq. mile, or 0.8 acre-ft. per acre, must be due to the decreased evapo-transpiration losses caused by the burning of the brush cover. The next season of heavy stream-flow record is 1921-22, which is also shown in the diagram. This curve approaches the line of equal run-off of both water-sheds, but is still on the Waterman Canyon side probably because of several small fires on that water-shed, which in the aggregate accounted for a considerable area of burned brush cover between 1911 and 1921.

In 1924 a fire burned over about one-half the Devil Canyon water-shed. A curve has been drawn from the stream-flow data since 1924. The Devil Canyon discharge has been corrected for diversions to the City of San Bernardino. This last curve falls on the Devil Canyon side of the line of equal flow. The differences between the curves for Waterman Canyon and Devil Canyon in Figs. 6 and 10 are due to the burning of the brush on the Waterman Canyon area. In this region where water is so valuable it appears to the writer that brush fires, which are the greatest menace to the timberlands in the higher area, and which excite the professional conservationist to hysteria and furnish the local newspaper with scare-heads rivaling those of some major action in the World War, are far from being an economic waste. All that remains is to put the excess run-off into the ground. Nature, by experimenting with this problem for thousands of years, has provided the débris cones or outwash slopes below the mouths of the canyons for this purpose.

The consumptive use of the brush cover on the outwash slopes is the determining factor in deep penetration of rainfall on the valley floor. Given the rainfall on the valley floor, and the consumptive use of the cover, the difference will be deep penetration. The run-off from the brush-covered valley floor is negligible. The author, after determining the requirements of the brush cover for the Devil Canyon water-shed on Fig. 8, ignores this factor when considering the valley floor area and attempts to solve the problem by utilizing his knowledge of seepage and storm-water escape from the mountain area and by applying it to the valley floor area. Table 10 is a set-up of this comparison.

The author uses his Waterman Canyon curve to obtain rainfall-run-off relations. His Devil Canyon curve would have been better suited to conditions, as it retained its natural brush cover until the fire of 1924. Column (3) of Table 10 shows the seasonal rainfall to be expected from storms, as given in Column (2). If the author had applied his consumptive use curve as shown in Fig. 8 to this seasonal rainfall, he would have found that there would have been left for deep penetration and run-off on the valley floor an average of 3 in., instead of 16 in., for the three storms given for the San Bernardino Valley in Column (2) of Table (10).

The author does not distinguish between the effect of storms occurring in the winter months of December, January, and February, when native chaparral growth is practically dormant, and the effect of storms during the months of March, April, and May, when the chaparral is making its most active growth due to the ideal conditions of long warm days and high soil moisture content. If he had determined the relation between the consumptive use of the chaparral on the mountain areas and that of the valley chaparral this question would have been taken care of in the comparison. The greatest difficulty in the way of such a comparison would be the effect of hold-over seepage water in the mountain areas after a wet season.

When the Department of Agriculture has completed its study of the consumptive use of native plants on the San Bernardino Valley floor, it may be found that this is not such an important item as the writer believes. The author has offered the basis for a comparison between consumptive uses of mountain and valley-floor brush, which can be used in the future without resorting to the elaborate continuous field investigations required by the Department of Agriculture. The writer is convinced that there is a very important contribution to the underground water supply in Southern California from deep penetration of rainfall on the valley floor, but he can not conceive of such a figure as 250 acre-ft. per sq. mile per year as an average for the brush-covered lands of the San Bernardino Valley. The writer believes that the greatest value of such a study was utilized when areas that were formerly brush-covered became cultivated with crops and orchards. As a result of this improvement the transpiration losses from the native growths that would draw on the rainfall were conserved and utilized for the growth of valuable crops and the creation of taxable wealth. Studies made by J. B. Lippincott, M. Am. Soc. C. E., in the Santa Ana Valley, and by Harold Conkling, M. Am. Soc. C. E., in the San Gabriel Valley, show that the substitution of cultivated crops for brush-covered areas has made very little inroad on the total water supply as it was conceived to be at the beginning of the Twentieth Century. The future benefits of any knowledge of rainfall penetration on the valley floor will be derived by increasing this quantity through removing the brush cover, or, substituting a shallower rooted growth. The brush on the mountain sides is absolutely essential and should not be destroyed, but there is the possibility of some systematic burning to remove the fire hazard and conserve excessive transpiration losses.

Determination of Penetration from Soil Moisture Tests on North Mesa, Murrietta-Temecula Area.—Table 4 shows the specific gravity, porosity, and probable yield of two soil samples at Test Plot No. 1, one from a depth of 2 ft., and the other from a depth of 4 ft. The increase in specific gravity, decrease in porosity, and decrease in probable yield for the sample from a depth of 4 ft. would indicate a marked change in soil structure. The author states that "the region is comparatively arid and the soil is an ancient, rather compact alluvium, a combination which would lead the casual observer to the conclusion that no deep penetration could occur." The writer made a casual observation of the region as shown on Fig. 4, and noticed a heavy hardpan underlying the top-soil wherever erosions had revealed the soil to a depth of 4 ft. The existence of hardpan is the best indication of the depth of rainfall penetration of which the writer has knowledge. The hog wallows and puddles mentioned by the author indicate that there is no free penetration of water, at least in the upper soil zone.

An examination of the data presented in Table 5 shows that the soil under Test Plot No. 1 is either very heterogeneous or that some artificial factors were introduced. Column (6) of Table 5 shows that the soil for all depths below the fifth foot was below field capacity on March 7, 1927. This fact would indicate that either the root zone of vegetation extends to a greater depth than 15 ft., or that the data given in Column (2) are not typical for the entire test plot. Any one comparing Columns (7) and (2) of Table 5, is led to believe that there was no deep penetration whatsoever, because the soil moisture at all depths to 15 ft. was less than the field capacity. The building of dikes or levees around the test plot would create a condition that would prevent the escape of moisture which otherwise might disappear as run-off.

The data presented by the fluctuations of wells given in Table 7 are far from conclusive evidence that the fluctuations were due to direct penetration of rainfall. The data for Well No. 129 shows that, despite the heavy rain in the season of 1926-27, the water level in July was lower than it was for the preceding year. There is no synchronism between the fluctuations of Well No. L8N and any of the others listed. The lack of frequency of measurements for Wells Nos. 131 and 14, which have the least depth to water, prevents any accurate study being made as to the effect of rainfall. These might be pressure fluctuations from distant sources, as both wells are near the obstruction which creates the springs and artesian wells mentioned by the author. If, as the author states, "there is corroborative testimony that the flow of the wells and springs varies little over a period of years, the inference being that the comparatively compact formation acts as a regulator of an otherwise erratic recharge," and there is no transmission of pressure, it is strange that Wells Nos. 131 and 14 should show a maximum rise of 1.21 ft. and 1.10 ft., respectively, when the maximum rises for Wells Nos. 129 and L8N were only 0.26 ft. and 0.29 ft., respectively. If these fluctuations were due to penetration of rainfall, then it must be assumed that two of the wells showed four times the penetration of the other two.

Determination of Rainfall Penetration from Test Wells in Pauba Valley on Temecula River, Riverside County, California.—The line of test wells used by the author in his determinations in the Pauba Valley extend from the mouth of an arroyo draining a part of the North Mesa to the Temecula River, as shown on Fig. 4. The author states that the storm of February, 1927, produced a rainfall of 14.65 in. in the Pauba Valley and, in Table 6, the same storm produced a rainfall of 9.14 in. at the test plot on the North Mesa. A rain of this intensity would certainly produce some run-off down the arroyo at the line of the test wells and add to the water supply. The author states that "in some instances the measurements indicated a rise due to run-off from adjoining mesa lands. On the other hand, there was a compensating run-off away from bottom-lands." The writer can not comprehend how run-off from the bottom-lands would compensate for water which would be absorbed in the valley from the run-off from the mesa lands. The Temecula River was undoubtedly in flood during the heavy rainfall period in February, 1927, and passing as it does, through the pervious valley fill of the Pauba Valley, would also contribute to the rise in the wells. The writer believes that the data presented by the author are far from conclusive as to the amount of contribution to the underground-water supply by deep penetration of rainfall on the valley floor.

This paper contains material some of which has been used in litigation proceedings.* From this and other sources the writer is led to question some of the points mentioned in the paper that should be recorded.

Fig. 4 does not show correctly the drainage of Long Valley. Test Plot No. 1 is located about 1½ miles too far south; the location shown is in a group of rough, broken hills.* The borings in the North Mesa Test Plot were made in duplicate and the samples averaged. Those portions of the samples not removed for office determinations were run back into the holes and tamped. The soil encountered in boring the holes ranged from a firm, semi-impervious soil to hardpan. The existence of the hardpan from 4 to 5 ft. below the surface shows the extreme depth to which rainfall has ever penetrated in this region. The tamping of the hardpan cuttings in refilling the test holes, a 2-min. operation, would leave a duct through which the ponded water in the test plot could percolate freely in comparison with the percolation through the undisturbed hardpan. The ponded area would prevent run-off escape. From Table 5 it appears that there were eight of these locally filled holes prior to the final determination on March 7, which would explain the presence of any excess moisture below 5 ft. under the test plot.

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* Rancho Santa Margarita *vs.* Vail, et al., Superior Court of San Diego (Calif.) County, Case No. 42850.

There is no general water-table underlying the mesas. The uncontradicted testimony in the case showed that suspended water-planes are found throughout the mesas, not by infiltration of rainfall on the surface, but by infiltration from water concentrating in gullies which cut the plain to contact with somewhat pervious lenses formed at intervals in the mesa silt.

The four wells submitted in Table 7 do not show true conditions. Near Well No. 129 is another one called Well No. 133, in which the water is usually from 65 to 75 ft. higher than in Well No. 129; near Well No. L8N is another called Well E7N, in which the water level is out of harmony with the one selected by the author and which shows a reverse slope from the river bed; near Well No. 131 is another one known as the "old Shrode Well", which has a water-plane from 40 to 50 ft. higher than Well No. 131, although down the valley where it should be lower, if in the same water-table; and near Well No. 14 are several wells and springs on the August Cantarini Ranch, which differ from it in water-plane elevation. All these facts, shown by sworn and uncontradicted testimony in Court, prove conclusively that there is no water-table under the North Mesa region and no penetration of rainfall to a water-table, but that suspended water-planes are found locally in some semi-porous lenses in the mesa silt where such lenses are cut into and fed by run-off water gathered in gullies.

Well No. L8N shows a drop by March 16, which is positive evidence that the slight rise of February 23 was caused by rain water running down outside the 2-in. pipe in this well. The well had been drilled by a rotary rig to a diameter of 5 in. in January, 1927, without casing, and a 2-in. standard lap-welded pipe inserted, the space between the outside of the 2-in. pipe and the inside of the 5-in. drill hole having been filled with loose soil. Of course, a little water would penetrate this loose fill from the heavy rains just before February 23, which would disappear before March 16. Well No. 131 had a depression in the ground around the casing which was 0.8 ft. above the bottom of this depression. There was much drainage from run-off into this depression. This caused the water to run into the well over the top of the pipe, whenever it became more than 0.8 ft. deep around the well, which did occur during the February, 1927, heavy rainfall. Further than this, the driller had tried out his perforating tool and had cut a number of slits in the casing just below the surface of the depression around the well, through which water was seen to enter during the February, 1927, rains.

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* Fig. 4 has been corrected, as shown in the paper, since Mr. Rowe's discussion was published.

The author assumes that the flow of the springs and wells is entirely derived from deep penetration of rainfall on the mesas. He ignores entirely the fact that there is a drainage area of more than 100 sq. miles of low granite mountains, foot-hills, and plains within the drainages of Santa Gertrudes Creek, Cantarini Wash, and Shrode Wash, which all converge at the natural dam formed by the Elsinore Active Fault. In each of these drainages are sand and gravel fans deposited as modern alluvium within erosions into the Pleistocene "mesa silt" extending back from the fault line. The floods from this large back-country drainage flow over these porous fans and are absorbed in part, after which they percolate toward the Elsinore Active Fault which forces at least a part of the water up to the surface as springs.

The situation in Pauba Valley is a familiar one to those acquainted with artesian basins of the interior California valleys. There is a deeply eroded trough in the mesa silt, which is refilled to a depth of more than 500 ft. with modern alluvium. This lies between the "North Mesa" and the "South Mesa", as shown in Fig. 4. Its lower end is closed by the Elsinore uplifted fault block and into its upper end debouches the Temecula River, draining a high, mountainous area of 326 sq. miles. This stream first eroded the mesa silt and then refilled the space with porous recent alluvium. It left a coarse fan or delta at the upper end and finer deposits farther down the Pauba Valley. The flood waters of the river spread over and are absorbed in part in the fan which is called the outwash area, and rise through artesian pressure in the lower valley, which is called the artesian area. When it rains on the mountain water-shed and the resulting flow sinks and refills the outwash area, hydraulic pressure, transmitted through the saturated inclined valley fill, causes the wells in the artesian area to rise. It was this rise which the author mistook for direct penetration of rainfall.

The author cites the record of only three test holes in the Pauba Valley. There were others introduced in evidence, which, after floods in the Temecula River, showed a greater rise than would have been accounted for, if the entire quantity of rain had penetrated to the water-table.

Ivan E. Houk,* M. Am. Soc. C. E. (by letter).—The results of the author's hydrologic studies are both interesting and valuable, especially in view of the fact that studies of deep rainfall penetration, or percolation, as it is more often termed, have seldom been made for irrigated sections of the country. His conclusion that "the water supply resulting from deep penetration of rainfall on the valley floors is of great economic importance", is true in the more humid States as well as in the semi-arid West.

In a general consideration of the disposal of rainfall it may be permissible to include deep penetration in lieu of seepage run-off and changes in ground storage. Deep penetration would equal seepage run-off in cases where the ground-water surface has the same elevation, and the moisture stored in the surface soil is the same, at the beginning and end of the time interval under consideration, assuming no water to pass beyond the limits of the basin by underground flow. In many drainage areas throughout the country, including gently rolling and comparatively level topography as well as mountainous regions, ground-water, or seepage run-off, may be appreciably different from deep penetration as far as yearly, or shorter, periods of time are considered. The author shows the importance of seepage flow in mountain drainage areas in his treatment of rainfall and run-off in the Devil Canyon Basin. Because of the importance of ground-water run-off it seems better, as a general rule, to consider the disposal of rainfall, as follows: (1) Flood, or storm-water, run-off; (2) ground-water, or seepage, run-off; (3) evaporation, including interception and transpiration losses, as well as evaporation from soil, water, snow, and ice surfaces; and (4) additions to ground storage, the last being positive or negative depending on whether the quantity of water in the ground has increased or decreased during the time interval considered.

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A discussion of the general conditions that affect deep penetration might well include some reference to the capillary moisture in the surface soil. Obviously, if this has been greatly depleted by evaporation and transpiration processes at the time the rain begins, a large storage reservoir will exist in the root zone of the soil, and this reservoir will absorb large quantities of water before much deep penetration takes place, even though fairly uniform wetting of the soil may not occur to the full depth of the root zone. The importance of this factor is clearly shown by the author's discussion of the soil-moisture tests made on North Mesa, in the Murrietta-Temecula Area. Furthermore, a consideration of capillary movements of soil moisture may be of some value in forming conclusions regarding the depth of uniform wetting of the soil. The author undoubtedly is correct in his conclusions† that "absorption is favored by the wedging action of roots," and that percolation tends to take place along well-defined ducts. However, it is quite likely that a fairly uniform wetting of the soil, to appreciable depths, if not to the full depth of the root zone, often takes place during the rainy season of the year, due to both capillary and gravity movements of soil moisture; and that in such cases the percolation of moisture through the minute passageways between soil particles is an important factor in conducting water to the well-defined ducts. Capillary movement of soil moisture may take place in any direction, horizontally, vertically upward, or inclined in either upward or downward directions, as well as vertically downward. Consequently, the vertical trickling streams at the ends of the suspended water-table, shown in Fig. 1, will occur only when the geologic formations below the horizontal clay layers are relatively open, as, for instance, in the case of coarse sand or gravel deposits. In the case of tighter formations the water at the ends of the clay layers will move horizontally and diagonally beneath the clay by both capillary and gravity action. However, in any case, there will undoubtedly be some deficiency of moisture immediately below the clay.

In studying percolation of rainfall it is important to know how fast water can enter different soil formations; and how the rates of absorption vary for a given soil, using the term, absorption, to include all water that enters the ground, percolation water as well as the capillary moisture taken up in the unfilled pore spaces of the soil. A clay formation, after it becomes wet enough to close the surface cracks by swelling of the material, will absorb water at a relatively low rate, if at all.* On the other hand a deep open gravel, or a sod surface underlaid at a comparatively shallow depth by open sand or gravel, will absorb rainfall at a relatively high rate. Experiments on a small plat in the Miami Valley of Southwestern Ohio showed that blue-grass sod, on level ground, underlaid by sand and gravel deposits at a depth of about 2 ft., would absorb water continuously at a rate of 13.0 in. per hour.† Soil deposits having rates of absorption of practically all values between these limits probably exist in various parts of the country. The maximum rate of absorption for a given soil may be somewhat lower in winter than in summer, due to the lower soil temperatures, the friction coefficients being higher for the lower temperatures. This effect is probably more pronounced in the case of relatively tight formations, than in coarse sand or open gravel deposits. However, in the colder regions, the maximum rate of absorption during the winter months is probably increased by the loosening of the surface soil, due to alternate freezing and thawing, much more than it is decreased due to the higher friction.

The experimental rainfall and run-off investigations in the Miami Valley showed that the maximum rate of absorption for soils in that vicinity, under ordinary conditions of compactness, becomes practically constant within a relatively short time after the beginning of the storm; that is, as soon as the surface storage becomes filled.‡ The surface storage includes storage space in the upper few inches of the soil; storage space formed by slight depressions in ground-surface level; and storage space on the ground surface formed by forest litter, accumulations of leaves, blades of grass, weed stems, and other forms of vegetation, both dead and alive. During the late summer and fall months, when the ground was comparatively dry, and the cultivated surface soils compacted by earlier rainfalls, from 3 to 4 in. of rain in 1 or 2 days were required to fill the surface storage. Flood run-off usually occurred sometime before the rate of soil absorption became constant, the length of time from the beginning of the storm until flood run-off began naturally being greater for the less intense rates of precipitation. Except in the case of the sod plats, underlaid at shallow depths by open sand and gravel, a rate of rainfall of 1 in. per hour caused flood run-off to begin in about 30 min., even on the dryest of soils; whereas a rate of rainfall of 3.00 in. per hour invariably produced flood run-off in less than 5 min. Rates of flood run-off were directly proportional to the rates of rainfall in all cases, regardless of the quantity of moisture in the soil. However, the actual value of the relation varied with the moisture content. In other words, the relation between the rate of rainfall and the rate of run-off could be represented by a straight line equation in all cases, but the actual value of the equation varied with the moisture content. For saturated soil conditions the average rate of absorption was approximately

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* "Water Penetration in the Gumbo Soils of the Belle Fourche Reclamation Project," by O. R. Mathews, U. S. Dept. of Agriculture, *Bulletin No. 447*, 1916, p. 4.

† "Rainfall and Runoff in the Miami Valley," by Ivan E. Houk, Miami Conservancy Dist. Technical Repts., Pt. VIII, Dayton, Ohio, 1921, p. 116.

‡ *Loc. cit.*, p. 124.

0.30 in. per hour. For soils loosened by spading, corresponding to freshly plowed fields, the rate of absorption was found to increase somewhat as the rate of rainfall increased, the relation being represented by the equation:

$$p = 0.37\, r + 0.16 \qquad \text{(1)}$$

in which, r and p are the rates of rainfall and absorption, respectively, both in inches per hour. Apparently, the loosening of the surface soil permitted the building up of a static head which increased with the rate of rainfall. A similar effect, but not so great, was found in the experiments on a hillside where the slope of the ground was about 18 ft. per 100 ft. Incidentally, one of the principal conclusions reached as a result of the investigations was that variations in surface slope are of much less importance as affecting flood run-off than variations in vegetable cover, or in the condition of the surface soil. In the experiment on spaded soil, water was applied at a rate of 3.73 in. per hour for 22 min. before any run-off took place.

The Miami Valley experiments herein discussed were made primarily for the purpose of studying flood run-off during intense storms. Rainfall was produced artificially, by sprinkling with ordinary garden sprinkling cans, and the total depths applied during the different experiments varied from 6.00 to 17.83 in. Each experiment was made up of several separate runs and the average rates of application during the different runs varied from about 0.2 in. to about 4.0 in. per hour. Soil formations at the locations of the plates were compact yellow clay tills, alluvial deposits underlaid by glacial tills, and yellow sandy loam underlaid by open sand and gravel. Complete descriptions of plats, as well as of methods of experimentation, results, etc., are included in the reference cited.

The all-important factor as regards percolation of rainfall on a given drainage area is undoubtedly storm rainfall. However, this factor is made up of two parts: First, the rate of rainfall during the storm; and, second, the duration of the storm. For instance, a storm with a total rainfall of 6 in. in 2 days will cause much more percolation if the rain is distributed uniformly through the 48 hours than if the greater part of the total falls in a relatively short time, say, 4 or 6 hours. When the rate of rainfall exceeds the rate of absorption an increase in the rate of rainfall means a higher rate of storm run-off and a lower total depth of absorption for the same total rainfall. Of course, an increase in the duration of the storm period, with the same rate of rainfall, means an increase in the total depth of absorption regardless of whether or not the rate of rainfall exceeds the rate of absorption. If the storm occurs after the surface storage has been filled the rate of absorption may be practically constant, and, in that case, the increase in total depth of absorption is directly proportional to the increase in storm duration.

The rate of absorption as herein discussed may, or may not, be wholly a rate of percolation. In the case of deep, open, sand and gravel deposits the water entering the ground would be practically all percolation, from the beginning of the rain to the end of the storm period. Consequently, the rate of absorption would be the rate of percolation throughout the storm. However, in most soil formations, the rate of absorption at the beginning of the storm would be definitely greater than the rate at which deep percolation would be taking place, the exact amount of the excess depending on the dryness of the surface soil. In the case of a comparatively shallow surface soil, underlaid by open sand or gravel deposits, subjected to an intense storm at the end of an extended dry period, the rate of absorption would include very little deep percolation at the beginning of the rain, but would include more and more percolation as the storm continued, approaching the maximum rate as a limit by the time the surface storage became filled, or within a comparatively short time thereafter. However, in the case of a drainage area with relatively deep soil deposits, it may, as the author states, "require a very wet year to recharge a deeply depleted soil." In such cases the rates of absorption would include but small proportions of percolation during the entire year.

The author's treatment of seepage and storm run-off in the Devil Canyon drainage area is pertinent and logical. However, the results, as shown on Fig. 8, are rather surprising inasmuch as the seepage run-off is such a large proportion of the total. The reason for the comparatively small storm run-off must be the dense surface cover of brush and trees. Similar storm rainfall on more open mountain water-sheds with similar surface slopes and similar geological formation, would probably cause much greater proportions of storm run-off. In fact, the writer is inclined to believe that the retarding effect of the dense vegetal cover is much more important than the accelerating effect of the comparatively steep ground slopes; and that, consequently, cultivated valley floors in that vicinity, subjected to the same storm rainfall, would have greater proportions of storm run-off, regardless of whether or not the underlying soil formations were clay. In this connection it may be valuable to refer again to the rainfall and run-off investigations in the Miami Valley of Southwestern Ohio, where similar studies of seepage and storm run-off were made.*

TABLE 17.—COMPARISON OF AVERAGE ANNUAL STORM AND SEEPAGE RUN-OFF, IN MIAMI VALLEY FOR YEARS 1915 TO 1919, INCLUSIVE.

Drainage area.	Area, in square miles.	Average rainfall, in inches.	Total run-off, in inches.	STORM RUN-OFF.		SEEPAGE RUN-OFF.	
				Inches.	Percentage of total.	Inches.	Percentage of total.
Miami River......	2 525	37.8	12.67	8.57	67.6	4.10	32.3
Buck Creek......	163	38.6	10.89	4.83	44.3	6.06	55.7
Mad River........	652	39.5	13.78	7.34	53.3	6.44	46.7
Stillwater River..	600	37.5	13.58	10.70	78.8	2.88	21.2

Table 17 gives a comparison of the average annual storm and seepage run-off for four Miami Valley drainage areas, for 1915 to 1919, inclusive. For those areas the average annual storm run-off is seen to vary from 44.3 to 78.8% of the total run-off, and the average annual seepage run-off, from 21.2 to 55.7% of the total run-off. The drainage areas are more than 90% agricultural land and may be classed as gently rolling topography with elevations varying from about 1 100 ft. above mean sea level near the head-waters to about 700 ft. above mean sea level at the bottom of the river in Dayton. The reason for the lower proportions of storm run-off in the Buck Creek and Mad River drainage areas is that the surface soil in those water-sheds is relatively shallow and is underlaid by open sand and gravel deposits, thus permitting comparatively rapid rates of percolation. In the Pomperaug Basin, in Connecticut, the annual ground-water run-off, for a 3-year period, was found to be 8.76 in., or 42% of the total run-off, the remaining 58% being storm run-off.* The Pomperaug Basin is a rural agricultural area of about 89 sq. miles, made up of rather rugged uplands and extensive valley areas, the elevations varying from a minimum of 100 ft. to a maximum of 1 150 ft. above sea level. Surface soil deposits consist primarily of glacial drift, although some of the steeper slopes are practically bare bed-rock.

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* Miami Conservancy Dist. Technical Repts., Pt. VIII, 1921, p. 164.

The author has secured some very interesting and valuable results in his exhaustive study of water supply from rainfall on valley floors. However, it is doubtful whether conclusions based thereon can be safely applied to other drainage areas, even in the immediate vicinity, without careful independent checking. This applies to conclusions regarding storm rainfall and flood run-off phenomena as well as to seepage run-off, percolation, and the resulting water supply. The geological, topographical, and meteorological conditions which affect rainfall, run-off, and percolation on different drainage areas are so many, so varied, and so variable that no intensive study of a single drainage area, or even of a group of drainage areas in the same locality, valuable as it may be in solving local problems, can furnish a satisfactory basis for general conclusions, or for the establishment of general laws universally applicable in studies of such phenomena.

Charles H. Lee,† M. Am. Soc. C. E. (by letter).—The author's treatment of deep penetration of rainfall is developed primarily for determination of the increment to water supply in the alluvial valleys of Southern California. The subject is one of considerable interest, however, in all semi-arid districts where growth of population has created a pressing demand for water. It may also be important in desert valleys, because the latter are usually surrounded by mountain ranges receiving more generous precipitation, a portion of which falls upon the marginal slopes of the valley. Although deep penetration from rainfall has been recognized as a source of sub-surface water, no definite methods have been generally adopted for quantitative determination on a large scale. The demand for such methods now exists, and this paper represents an initial step in filling this need.

At the outset, in the consideration of the paper, it should be recognized that the character of alluvial deposits as generally found in valleys does not conform with the author's description and illustration on Fig. 1. It is true that in many of the alluvial valleys of Southern California there are deposits which "are essentially heterogeneous in formation". These are the products of deposition from steep graded streams and result from reduced velocity due to flattening of slope. For these deposits the author's concept may be correct.

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* "A Study of Ground Water in the Pomperaug Basin, Connecticut, with Special Reference to Intake and Discharge," by Oscar Edward Meinzer and Norah Dowell Stears, U. S. Geological Survey, *Water Supply Paper 597-B*, p. 115.

† Cons. Hydr. Engr., San Francisco, Calif.

There are many other alluvial filled valleys and parts of valleys, however, throughout California and, in fact, generally throughout the world, in which homogeneity in formation is the ruling characteristic. Such valleys at one time or another in their history have experienced temporary flooding, or have been occupied by large lakes or arms of the sea which have had wide fluctuation in level. In these valleys, stream deposition has resulted either from the sudden checking of velocity at the edge of standing water, or from flattening of grade at the margin of an unwatered lake bed. These conditions have resulted in segregation of alluvial material into extensive sheets or beds of distinctive character. The beds consist of fine sands, silts, and clays, deposited from still water upon the lake bottom, and coarse sands and gravel derived from rehandled and segregated shore deposits which have been spread over the lake bottom by flowing water at periods of low lake stage. The occurrence of homogeneous valley formations is so general that the discussion of the paper would be incomplete if mention were not made of them.

Another concept presented by the author, which is stated without qualification, is that because of the heterogeneity of alluvial deposits the movement of absorbed water from the ground surface to the water-table is by way of "more or less defined ducts", and that it is "likely to collect into trickling streams".

Although the condition described by the author may be characteristic of the steep boulder and gravel slopes surrounding certain Southern California valleys, yet it is not a general condition. Most valleys have a soil cover which is of homogeneous texture and is underlaid by a subsoil of similar texture. In such formations the downward movement of water is generally through capillary interstices, and is more or less uniformly distributed. It frequently occurs that roots will open up a soil and, when rotted, will leave tubes which assist in the more rapid downward penetration of water; but this is merely an aid to penetration, and in no case does it extend to great depth.

The mechanics of rainfall penetration are so important to an understanding of the problems considered by the author, that the writer believes a short description would not be amiss. If rain falls upon soil, with moisture content at or exceeding field capacity, there will be an immediate gravity movement of water by way of fat capillary films to the water-table, and an observable rise of the latter. If, however, the soil has been depleted of moisture below the field capacity, penetration to the water-table does not occur unless and until complete connection has been established by capillary films extending from the surface to the water-table.

As the advance of moisture by capillarity alone is extremely slow, it frequently happens, in regions where the annual depth of rainfall is small as compared to the annual depth of water evaporation, and in areas where the depth to the water-table exceeds the capillary range, that little or no water will reach the water-table during a whole season. The reason for this is that the losses by run-off interception, soil evaporation, and transpiration are so great that most or all water falling as rain is consumed either before or after penetrating the soil.

This point is well illustrated by observations made by the writer at twenty-four wells in the river valleys of San Diego County, California, before and after the first heavy storm of the season, 1914-15, beginning January 27, 1915.* The formation in these valleys is of uniform textured beds of fine to coarse sand, the top stratum containing more or less silt. The soil and sand above the capillary fringe was very deficient in capillary moisture at the end of nine months' rainless period. It was found that at all wells in which the depth to the water-table, just prior to the storm, was less than the distance to which capillary water would rise (approximately 7 ft.), the effect of the storm on the water-table was pronounced and reached its maximum within from 2 to 7 days. For wells in which the depth exceeded 7 ft., no pronounced fluctuation was observed although after a short time had elapsed a gradual rise was noted culminating in from 20 to 60 days. The range of depth to water-table in the latter wells was from 7.4 to 19.4 ft. A very sharp difference was evident, and there was no difficulty in grouping the wells.

Comprehensive observations have been made by Walter E. Spear, M. Am. Soc. C. E., of the deep penetration from rainfall in rehandled glacial deposits of Long Island, New York, where the soil and subsoil are maintained more or less at field capacity throughout the year by frequent storms.† More than fifty wells were studied, with widely differing depths to water-table, which ranged from 1 ft. to 95 ft. The material through which moisture percolated consisted of 12 to 18 in. of brown loam, 18 to 24 in. of yellow subsoil, and glacial outwash made up of coarse to fine sand occasionally mixed with coarse and fine gravel, and very fine sand. For depths to water-table of 7 ft. accretions from rainfall were registered at from 1 day to 4 days, which correspond closely with the writer's results in San Diego County. For a depth of 20 ft., the time interval was from 4 to 14 days. In comparison with the latter, the time required in San Diego County wells with deficient capillary moisture was from 20 to 60 days. The effectiveness of soil moisture content at field capacity in facilitating deep rainfall penetration is thus clearly apparent.

The importance of regular sequence and magnitude of storms in producing deep penetration in regions with distinctive wet and dry periods, which has been so well brought out by the author in his discussion of general conditions, is further emphasized by the data contained in Mr. Spear's report. Depletion of soil moisture always occurs during long periods without rain, unless a permanent water-table lies within capillary distance of the surface. In an area where the water-table lies more than from 7 to 10 ft. below the surface, a certain volume of water is required to satisfy capillary demands before penetration to the water-table will occur. This volume, and the time required, increases with the depth to the water-table. In order that penetration to the water-table shall occur, the rainy season must be long enough, and the storms sufficiently frequent and large to bring the moisture-depleted soil and subsoil up to field capacity from the surface to the water-table prior to the end of the season. The extent to which moisture reaches the water-table through "well-defined ducts" or "trickling streams" is under ordinary conditions extremely problematical, and the writer believes cannot be depended upon to speed up the process except in rare cases, such as that which the author seems to have encountered.

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* *Water Supply Paper 446*, U. S. Geological Survey, 1919, by A. J. Ellis and Charles H. Lee, p. 140, Table 35.

† Report of Commission on Additional Water Supply for the City of New York, 1903, Appendix VII, by Walter E. Spear, Plate VI and Table 25, pp. 798 to 805.

Determination of Rainfall Penetration from Soil Moisture Tests and from Springs in the Murrietta-Temecula Area.—The author assumes, for the purpose of reaching quantitative conclusions, that deep rainfall penetration from a certain mesa area underlaid by compact stratified alluvium is the only source of supply for a group of springs and flowing artesian wells. The latter are located at the base of a steep slope terminating the mesa. To prove that deep penetration occurs, he presents soil moisture records at test plots which are enclosed by embankments to prevent surface run-off. These records indicate that of 14.82 in. depth of rainfall and water applied to the plots between February 4 and March 7, 1927, 3.31 in. were unaccounted for within the first 9 ft. depth of soil, if evaporation loss from surface soil is assumed at 2.00 in. (see Table 5). The author concludes that this water penetrated deeply and reached a water-table approximately 200 ft. below the surface of the mesa. As proof, he presents water-level observations in four wells, three of which show a slight rise in the late spring of 1927 (see Table 7). He presents no proof, however, that there is a general water-table under the mesa, or that the springs and artesian wells have any relation to a general water-table.

The writer believes that in a few special cases this method may be useful in obtaining quantitative data of deep percolation, but that it must be used with caution. In the example presented by the author, there are several points on which the facts submitted are incomplete or not in conformity with the conclusions. For instance, assuming that percolation to a depth greater than 9 ft., occurs from a few isolated depressions on the surface of the mesa, does it occur generally over the whole area, and is all water penetrating below 9 ft. beyond the reach of capillarity and plant roots? The well observations presented in Table 7 are not conclusive on this point. Well 129, near the test plots, does not show any rise. Well L8N, farther down the slope, shows 0.25 ft. rise in 27 days; and Wells 131 and 14, still farther down the slope, show rises of 1.30 ft. in less than 90 days and 1.10 ft. in 47 days, respectively. Thus, the greatest rise occurred at the lower edge of the slope of the assumed water-table, and a profile would show a flattening of the slope during the period when the author assumes that it is being replenished by deep percolation. Such flattening is, of course, an indication of depletion rather than of replenishment.

Considering another angle, is it an entirely reasonable assumption that water would move downward in less than 90 days through 200 ft. of moisture-depleted material composed of compact stratified alluvium containing extensive clay beds? In the test made by Mr. Spear on Long Island where conditions are most favorable for quick penetration, it required from 89 to 109 days for moisture to reach a depth of 80 ft. Is it not a more probable assumption that, if penetration occurred beyond the depth of capillary return, the water would move laterally as soon as it reached an impervious clay bed, and escape into some ravine or follow an old buried stream channel to the margin of the alluvium? The observed rise of water level in wells might possibly have been a rise due to pressure increase and not a water-table rise.

The author does not present the data upon which he determines the area from which the flow of the springs and wells is derived. Due to the varied structure of the alluvial formation, this would be subject to considerable uncertainty unless fairly complete geological and hydraulic facts were available. It is quite possible, for example, that the springs and wells represent the discharge from the open end of an old buried stream channel interbedded between clay strata instead of from a general water-table. If this is the case, the tributary area is impossible of determination from an examination of surface topography alone. The fact that there is not an extensive spring or seepage horizon along the base of the slope adjacent to the springs would support such an assumption.

The uncertainties, previously mentioned, indicate some of the difficulties involved in the practical application of this method, and they point to the fact that the author's quantitative result, while suggestive, is not the final answer.

Determination of Rainfall Penetration by Comparison with Mountain Run-Off.—The method of determining rainfall penetration by comparison with mountain run-off is based on two assumptions: First, that for mountain water-sheds underlaid by impervious bed-rock which extends to the point of stream measurement, surface run-off throughout the annual cycle will correspond to the sum of surface run-off and deep percolation in adjacent valley water-sheds; and, second, that rainfall-run-off relations for adjacent mountain and valley water-sheds are so similar that necessary modifications due to differences in soil cover, etc., can readily be made.

The writer believes that this method has possibilities for general use, but that its successful application will depend upon quantitative evaluation of the various factors which govern the rainfall-run-off relation, rather than dependence on geographic proximity of water-sheds.

The factors upon which the rainfall-run-off relation depend, can be grouped as meteorological and physical. Among the important meteorological factors are: (1) Precipitation; (2) temperature; (3) evaporation from water and snow surfaces; and (4) length of the growing season.

The physical features of water-sheds, which are of importance, are: (1) Elevation; (2) topographic catchment area; (3) topography; (4) soil and subsoil; (5) geology; (6) vegetation; and (7) cyclic storage capacity.

The precipitation falling upon a water-shed is disposed of either by: (1) Storm or seepage run-off, the latter being represented by deep percolation in valley water-sheds; (2) evaporation from water intercepted by foliage and branches of vegetation; (3) evaporation from snow or from soil moistened by direct precipitation, melting snow, or capillary water held in the soil; (4) transpiration from growing vegetation; or (5) sub-surface leakage. Run-off curves, when prepared quantitatively in terms of depth of precipitation and run-off, as in Fig. 8, segregate these various items into two groups, as follows:

For Mountain Water-Sheds:

1.—Storm and seepage run-off.
2.—Evaporation, transpiration, sub-surface leakage (usually negligible).

For Valley Water-Sheds:

1.—Storm run-off.
2.—Evaporation, transpiration, seepage run-off, or sub-surface leakage.

If the valley occupies a closed basin, and storm run-off is measured at the outlet, sub-surface leakage will be small or negligible, represented by the underflow of the stream. For open valleys, sub-surface leakage may be large, although it does represent a definite quantity of water that can be salvaged by pumping from wells and, therefore, may be considered as potential water supply.

The quantitative rainfall run-off curve, for any water-shed, presents graphically the integrated effect of the four meteorological and seven physical factors listed previously. From this curve can be measured, for any depth of annual precipitation, the corresponding depth of evapo-transpiration loss plus sub-surface leakage; minor corrections are sometimes necessary for cyclic storage. For mountain and closed valley water-sheds this quantity represents the evapo-transpiration loss alone. Its value has one of two absolute upper limits, either the annual depth of evaporation from a large free water surface (if that is less than the annual depth of precipitation), or the annual depth of precipitation, if the latter is less than the evaporation. Actually, the evapo-transpiration loss is less than either of these quantities, being limited by the evaporation opportunity.

In order to give perspective in analyzing the weight to be assigned various factors which influence the evapo-transpiration value for mountain water-sheds, the writer has found it useful to study water-sheds with widely differing meteorological and physical characteristics. As an illustration, the rainfall run-off curves of five mountain water-sheds are presented on Fig. 15. A statement of the corresponding water-shed characteristics is given in Table 18, and the trunk stream profiles are shown on Fig. 16.

The values for Item 15 of Table 18 were determined graphically from Fig. 15, by making necessary corrections to allow for the actual slope area. The temperatures given in Table 18 (see Items 8, 9, and 10) are based on U. S. Weather Bureau records obtained at stations representative of the water-shed area. Item 16 for the different water-sheds was determined in various ways, as follows:

(*a*) For Coquille River an estimate was made from an incomplete 4-year record obtained by the U. S. Weather Bureau at Corvallis, Ore.

(*b*) The Rogue River evaporation was estimated from a 4-year floating-pan record at Klamath Falls, Ore.* (Elevation 4 100), and from an incomplete 4-year record at Fish Lake, Oregon† (Elevation 4 687).

(*c*) The average annual precipitation for Devil's Canyon Creek was estimated as approximately 55 in.

(*d*) The corresponding value for the San Luis Rey River is the average of a 3-year floating-pan record at Big Lake, near Warner's Hot Springs, Calif.‡ (Elevation 2 780).

(*e*) For the San Diego River, an average was taken from the 3-year floating-pan records at La Mesa Reservoir (Elevation 480) and Cuyamaca Reservoir§ (Elevation 4 620).

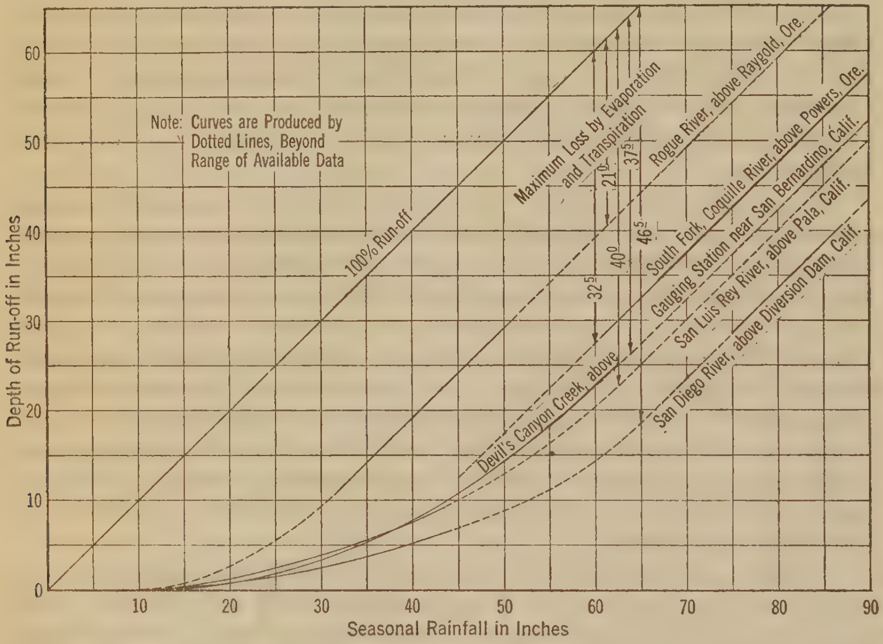

FIG. 15.—COMPARISON OF RAINFALL-RUN-OFF CURVES FOR STREAMS IN SOUTHERN OREGON AND IN SOUTHERN CALIFORNIA.

For the South Fork of Coquille River and for Rogue River in Southwestern Oregon, precipitation equals or exceeds evaporation; while for San Luis Rey and San Diego Rivers, and also for Devil's Canyon Creek in Southern California, the reverse is true. Precipitation occurs largely as snow on the Rogue River, while snow falls only upon the higher parts of the other water-sheds. The bed-rock formations also differ. Sedimentary rock underlies the Coquille, while the Rogue is largely covered by deep lava flows resting upon crystalline rock. The Southern California water-sheds are of granitic formation. Wide differences in vegetational cover also exist. The Coquille supports a very dense forest growth, largely Port Orford cedar and Douglas fir, with alder and luxuriant shrubbery and undergrowth filling open spaces; the Rogue has open forest cover of pine and fir; and the Southern California streams have dense brush cover with alder, sycamore, and willows along stream bottoms.

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* *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 273.

† U. S. Weather Bureau, Climatological Data, Oregon Section, Years 1923 to 1928.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1909.

§ *Water Supply Paper 446*, U. S. Geological Survey, p. 102.

Precipitation data for the preparation of all rainfall run-off curves have been obtained by planimeter measurement from detailed isohyetose maps of average seasonal precipitation corrected in each year by the index of seasonal wetness as derived from five or more base precipitation stations. Most of the latter have twenty-five years or more of record. Run-off data are available for periods of 10 to 25 years on the various streams. All precipitation, run-off, and drainage-area data are from the records of the U. S. Weather Bureau and U. S. Geological Survey. The curve for Devil's Canyon Creek as thus drawn (Fig. 15), differs from that of the author (Fig. 8). The principal reason is that the writer used a greater depth of average seasonal precipitation which seems to be supported by all available records.

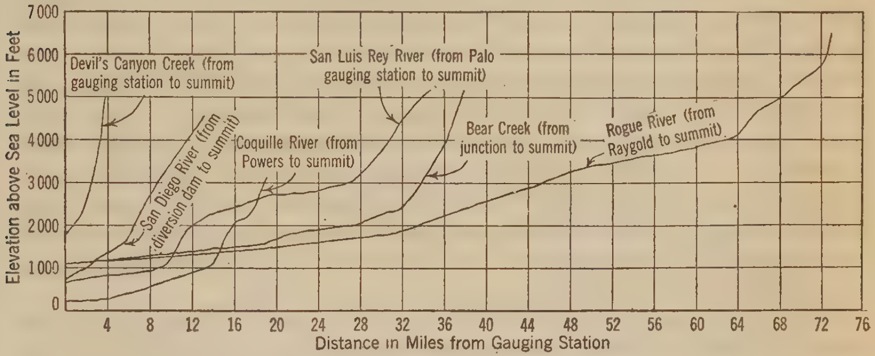

FIG. 16.—PROFILES OF DRAINAGE AREAS FOR STREAMS IN SOUTHERN OREGON AND SOUTHERN CALIFORNIA.

A striking feature of the curves for the two Oregon streams is their parallelism with the 100% run-off line within the full range of observed data, including both very wet and very dry years. (Compare the solid lines; the curves are produced by dotted lines beyond the range of available data.) This is due to the fact that even the minimum seasonal precipitation exceeds the maximum evapo-transpiration demand. For Devil's Canyon watershed the upper end of the curve is just coming parallel to the 100% run-off line at the 60-in. rainfall, indicating that the precipitation of wettest seasons is just sufficient to meet maximum demands of evaporation and transpiration. For San Luis Rey and San Diego Rivers, maximum seasonal precipitation is far too small to satisfy the demand, and the upper ends of the curves (at 45 in. of rainfall) have angles of departure with the 100% line. The condition necessary for maximum evapo-transpiration loss to occur from these water-sheds is greater annual precipitation and a regular succession of storms during the wet season at intervals just necessary to keep the soil at field capacity. On Fig. 15, the five rainfall-run-off curves have been assembled, and with the assumption of maximum conditions, the San Luis Rey and San Diego curves have been extended in conformity with the trends of the upper ends of the supported portion of the curves and with a general shape similar to the Devil's Canyon curve. The maximum annual evapo-transpiration loss for each of the five streams as scaled from Fig. 15, together with the annual evaporation from a large free water surface and computed evaporation opportunity ratio, are listed as Items Nos. 15, 16, and 17, respectively, in Table 18.

"Evaporation opportunity" is a very useful coefficient because it represents quantitatively the ratio of maximum evapo-transpiration loss for a given combination of meteorological and physical conditions to the evaporation from a large free water surface for the same meteorological conditions. Analyzing the values in Items Nos. 15, 16, and 17 of Table 18, the Coquille, with a coefficient of 0.90, shows the combined effects of maximum interception of rainfall by dense vegetation, the maximum holding of moisture within the root zone by vegetation mat and humus soil, and unrestrained transpiration from a 100% vegetation cover. The Rogue, with a coefficient of only 0.47, registers the effectiveness of lava in absorbing moisture from melting snow and transmitting it downward below the root zone, and also the more open character of the forest cover and the sparseness of vegetation in the valley area. The three Southern California water-sheds have physical characteristics quite similar to the Coquille, with the exception of vegetation type and density. The coefficients average 0.66. This value is less than for the Coquille principally because of difference in the extent of interception by vegetation, and the difference in transpiration loss, the latter due both to differences in rate and in density of vegetation.

The analysis and comparison of the complete curves for these five streams suggest a possible method for logical construction of run-off curves for mountain water-sheds for which run-off data are incomplete. The control for constructing such curves is comprised largely in determining values for: (1) The minimum rainfall necessary to produce run-off from the water-shed; (2) evaporation opportunity; (3) annual evaporation from large free water surface for the meteorological conditions of the water-shed; (4) the vegetational tolerance with respect to variations in water supply; and (5) the storm distribution of seasonal rainfall.

(1) The minimum rainfall necessary to produce run-off from the water-shed determines the point of beginning of the curve, and for mountain water-sheds it varies from 5 to 10 in., depending upon water-shed characteristics and the index of wetness.

(2) The ratio, evaporation opportunity, varies for different water-sheds from 0.40 to slightly more than 1.0, depending largely upon the physical conditions and vegetation cover. If the annual evaporation from a large free water surface and the evaporation opportunity are known, the maximum evapo-transpiration loss can be computed. This, in turn, makes it possible to establish the position of the straight-line section of the run-off curve with respect to the 100% run-off line.

TABLE 18.—WATER-SHED CHARACTERISTICS FOR TWO OREGON AND THREE SOUTHERN CALIFORNIA STREAMS.

Item No.	Name.	South Fork, Coquille River, above Powers, Ore.	Rogue River, above Raygold, Ore.	Devil's Canyon Creek. above gauging station. near San Bernardino, Calif.	San Luis Rey River, above Pala, Calif.	San Diego River, above "Diverting Dam," Calif.
15	Maximum evapo-transpiration loss, in inches.........	32.5	21.0	37.6	40.0±	46.5±
1	Elevation above sea level, in feet: Mean.......	......	......	3 600	......	......
2	Maximum..	3 200	6 500	5 250	5 250	4 500
3	Minimum ..	220	1 100	1 750	650	750
4	Area. in square miles.......	168	2 020	6.16	322	90
5	Average seasonal precipitation: Character.................	Rain with occasional snow at higher levels.	Snow with rain at lower levels.	Rain with occasional snow at higher levels.	Rain with occasional snow at higher levels.	Rain with occasional snow at higher levels.
6	Amount, in inches........	96.0	43.6	34.3	25.6	27.8
7	Seasonal distribution; 75% occurs in the period.	November 1 to March 31.	November 1 to April 30.	December 1 to March 31.	November 1 to March 31.	November 1 to March 31.
	Temperature, in degrees Fahrenheit:					
8	Mean annual..............	52.0	48.0	......	58.6	56.2
9	Maximum monthly.......	62.9	66.6	......	73.7	69.3
10	Minimum monthly........	41.4	31.4	......	44.2	44.2
16	Average annual evaporation from large free water surface, in inches............	36±	45±	55±	62.2	71.0
11	Topography..............	Mostly steep slopes, low relief in flats, and ridge tops at head of drainage.	Moderate slopes in mountains and foothills; flat slopes in valleys (8 per cent.).	Steep slopes.	Largely steep slopes with central portion occupied by large low relief valley (15 per cent.).	Mostly steep slopes, small low relief valleys at head of drainage.
12	Soil........................	Shallow residual with much humus and forest litter.	Decomposed lava and pumice in mountains: shallow residual in foothills; alluvial in valley.	Shallow residual with decomposed granite subsoil.	Shallow residual with decomposed granite subsoil, clayey soils in valley.	Shallow residual with decomposed granite subsoil.
13	Geological formation.....	Early tertiary sandstone, shale, and conglomerate.	Mostly lava with crystalline rocks and shallow valley fill in lower portion.	Granitic with some old alluvium flats and detrital slopes.	Granitic with lacustral deposits occupying valley portion.	Granitic.
14	Vegetation: Character and density during the growing season..........	Dense forest cover of coniferous and deciduous trees with leafy underbrush.	Coniferous forest cover in mountainous portion, foothills and valley, brushy or bare.	Dense brush cover with alder and sycamore in stream beds.	Dense to thin cover on slopes with oak and pine at higher levels, valley margin and floor bare.	Dense brush cover with sycamore and willows in stream bed.
17	Evaporation opportunity...	0.90	0.47	0.68	0.64	0.66

(3) Annual evaporation should be as observed in a floating submerged pan exposed on a large free water surface, such as a lake or reservoir. Its ordinary range of variation is from 30 to 72 in.

(4) Vegetational tolerance with respect to variations in water supply, has considerable influence on the shape and location of the point of tangency of the straight line and the curved part of the run-off curve. The more tolerant the vegetation, the flatter the curve and the greater the depth of run-off at which the change from curve to straight line occurs. Deciduous and coniferous trees are normally very exacting in their water requirements. Brush cover is quite tolerant. The most flexible types of vegetation are the annuals, such as flowers, weeds, grasses, tules, etc.

(5) The storm distribution of seasonal rainfall has a direct bearing on total loss by interception and soil evaporation. Maximum conditions occur for a regular sequence of storms throughout the wet season at intervals of proper length to keep soil moisture at field capacity. Its relation to the run-off curve corresponds with that of vegetational tolerance.

The principles which govern valley run-off plus deep percolation are essentially the same as for mountain water-sheds, and it would appear that the foregoing method could be extended to valley water-sheds with appropriate modifications. The item of greatest uncertainty, because of the many factors influencing it and the lack of digested data, is the evaporation opportunity. Factors which would tend to make this ratio larger for valley than for mountain water-sheds are:

(*a*) The usual absence of shallow sloping bed-rock in valleys, which in mountain water-sheds tends to hold percolating water within the root zone of vegetation.
(*b*) The greater intensity of storms in the valley, at least in Southern California.
(*c*) Lower density of natural vegetation cover in a valley, which affects both interception of rainfall and the transpiration aggregate. The effect of Factor (*c*) may be somewhat offset by increased exposure of the soil to evaporation.

Factors that tend to make the ratio larger are: (1) Flat topography in valleys instead of broken, as on water-sheds; (2) cultivation of valley lands; and (3) in the valley, greater depth below the ground surface to which moisture depletion less than field capacity may occur.

Referring to Factor (1), the actual land area of a mountain water-shed may exceed the projected map area by from 10 to 20 per cent. If the conversion of measured run-off from volume to depth is made on the basis of actual land area, the result will be correspondingly reduced. This may result in an increase of evaporation opportunity for valley water-sheds of from 5 to 10 per cent. Cultivation of valley lands (Factor (2)) offers greater opportunity for absorption by the soil during storms and thus greater loss by soil evaporation between storms.

In the absence of actual data for the evaporation opportunity ratio for valley water-sheds, assumptions can be made based upon the ratio in adjacent mountain water-sheds with modification as comparison of influencing factors suggests. A run-off curve may then be constructed following the outline described herein and using values for other items which ordinarily are available. The resulting run-off curve includes deep percolation and storm run-off. The former can then be segregated by constructing a storm-water run-off curve from measurements of valley run-off, and taking off of differences.

The writer has applied this method to the determination of penetration in mountain water-sheds, and because of its practical usefulness in constructing mountain run-off curves, he believes that its use may be extended as future determinations of evaporation opportunity for type valley water-sheds are made.

Determination of Rainfall Penetration from Test Wells in Pauba Valley.—The method of determining rainfall penetration from test wells in Pauba Valley, as described by the author, has distinct possibilities where it can be applied free from influences other than precipitation. During the winters of 1913-14 and 1914-15 the writer had under observation more than 120 wells and test holes in various valleys of San Diego County, California, many of which were located in valleys of a character similar to the Pauba Valley. This experience led to caution in the interpretation of water-level observations under the conditions such as exist in the Pauba Valley. One of the sources of difficulty was the influence of flood run-off in an adjacent stream upon water level in test holes in the same alluvial formation. This interference may occur either as a water-table "wave" traveling slowly outward from the stream channel, or as a quick transfer of pressure through a stratum of coarse sand. The former would not produce sudden rises, such as are indicated on Fig. 5, unless test wells were located close to the channel. It is possible, however, that such rises were the result of pressure transfer not reflected in the water-table. In the case of the three wells shown on Fig. 5, the possibility of a pressure effect could be easily determined by ascertaining whether flood run-off in Temecula River occurred during the storm of December 3 to 10, 1927. During this storm nearly 9 in. of rain fell and there was no sudden rise in water level in the wells, due to the moisture deficiency of the sand.

The author omits to state his method of determining the factor used in converting observed rise of water-table to equivalent depth of water. He states that "a porosity factor of 25% was allowed". The writer has frequently had occasion to make this evaluation, and has developed a graphical method which takes into account existing capillary moisture above the water-table before rise commences. Apparently, the author has not given attention to this phase.

The method is based on the fact that the volume of water in a soil column of given depth and unit area is equal to the average percentage of moisture by volume in the column multiplied by the depth. Hence, on a moisture percentage-depth graph (see Fig. 17), the volume of moisture in any column is represented graphically by the area between the moisture curve and the line of zero moisture. The volume of water represented by a given rise in the water-table, is equal to the difference of the water in the soil between the original water level and the surface of the ground, after and before the rise occurred. This volume is graphically represented by the area between the moisture percentage curves before and after the rise. For ordinary conditions these curves are similar. The following is the development of a simple equation for computing the volume. Since a soil column of unit area is assumed, volume and depth have the same value. For a soil column 1 sq. ft. in cross-section, extending from the ground surface to the lowest level of the water-table (Fig. 17):

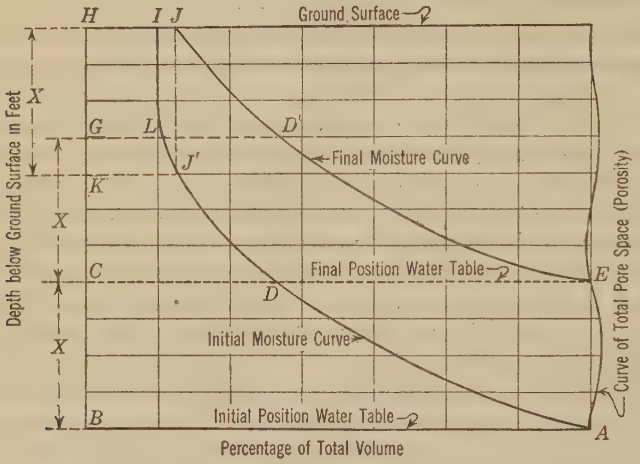
H
I J
Ground Surface
Depth below Ground Surface in Feet
X
G
L
D'
J'
K
Final Moisture Curve
X
C
Final Position Water Table
E
D
Curve of Total Pore Space (Porosity)
Initial Moisture Curve
X
B
Initial Position Water Table
A
Percentage of Total Volume

FIG. 17.—DEVELOPMENT OF FORMULA FOR CONVERTING OBSERVED RISE OF WATER-TABLE INTO EQUIVALENT DEPTH OF WATER.

Let v = volume or depth of water represented by a rise, X, of water-table;

r = volume or depth of water represented by the total pore space of soil through which water-table rises; and

c = volume or depth of water in the soil between the ground surface and a level, X ft. below the ground surface.

Then, because of geometric relations,

$$v = \text{Area } ADE + \text{Area } EDLIJD'$$

$$r = \text{Area } ADE + \text{Area } ABCD = \text{Area } ADE + \text{Area } ECGD'$$

$$c = \text{Area } J'KHI = \text{Area } HILG + \text{Area } GLJ'K = \text{Area } HILG + \text{Area } DCGL - \text{Area } DCKJ'$$

Hence,

$$v = r - \text{Area } ECGD' + \text{Area } EDLIJD'$$

but,

$$\text{Area } EDLIJD' = \text{Area } ECGD' - \text{Area } DCGL + \text{Area } D'GHJ - \text{Area } HILG$$

Substituting,

$$v = r - \text{Area } DCGL + \text{Area } D'GHJ - \text{Area } HILG$$

and since,

$$\text{Area } D'GHJ = DCKJ'$$

therefore,

$$v = r - c \dots\dots\dots\dots\dots\dots (2)$$

In other words, the volume of water represented by a rise of X ft. in the water-table equals the total pore space in the soil column through which the water-table rises, minus the moisture in the soil column between the ground surface and a level X ft. below the ground surface.

In the use of Fig. 17, the average percentages for total porosity in the column X ft. above the initial position of the water-table, minus the average percentage of moisture on the column X ft. below the ground surface, multiplied by the rise in the water-table, in feet, gives the equivalent depth of actual water, in feet.

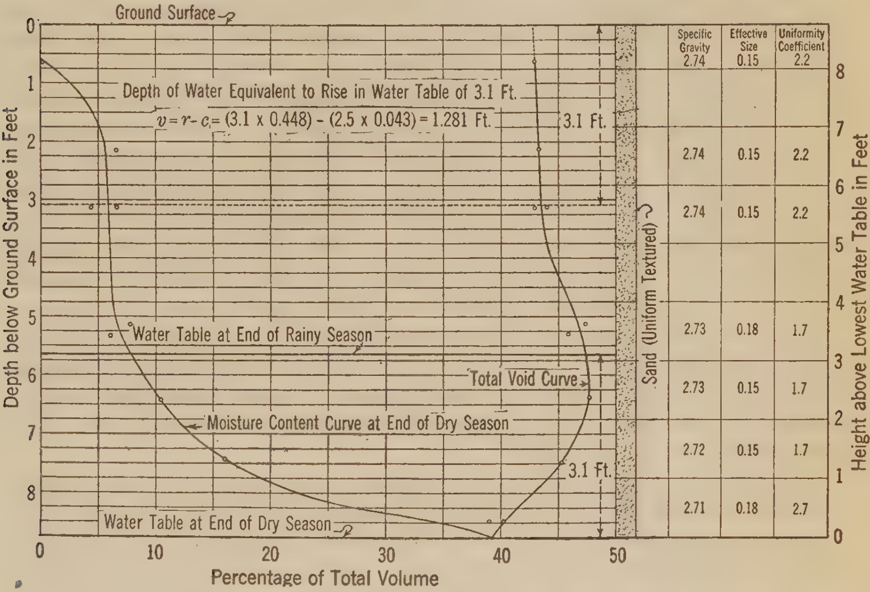

FIG. 18.—APPLICATION OF FORMULA FOR CONVERTING OBSERVED RISE OF WATER-TABLE INTO EQUIVALENT DEPTH OF WATER.

In applying the method, type areas are selected such that the character of the soil and subsoil is more or less uniform. In each area a pit is dug to the water-table at the end of the dry season. Samples are cut from the exposed face, at intervals of 1 ft., or more, so as to be representative of the gradations of material in the column, and initial moisture and total porosity are determined. If desired, a mechanical analysis of each sample may also be run, although this is not necessary. The data are then prepared in graphical form, as illustrated by Fig. 18, which represents the condition where the depth to water-table exceeds the capillary limit of the material. This diagram represents a typical material for similar valleys in this region, including Pauba Valley. It is to be noted that the computed depth of equivalent water represents percentage by volume of 39.4 for a material with an average porosity of 45 per cent. It is probable that the author's assumption of 25% for net porosity is too low.

A. L. Sonderegger,* M. Am. Soc. C. E. (by letter).—The extensive discussion of this paper is proof of the interest which the subject has aroused of late years in the semi-arid West. It has been fruitful of many new and interesting ideas and facts and not a few contradictory statements; also, some criticism. A discussion of natural phenomena, such as rainfall penetration and run-off, and particularly of the methods for their determination which the paper was intended to bring out, must, of necessity, make reference to specific cases.

Mr. Houk, in the closing paragraph of his discussion, summarizes the general conclusion reached, namely, that the geological, topographical, and meteorological conditions which affect rainfall, run-off, and percolation, are so many, so different, and so variable that general conclusions, or the establishment of general laws, cannot be based upon studies of an individual area or a group of areas. The discussion has brought out that present knowledge of the many factors which affect the disposal of rainfall is only fragmentary. There is little or no information available as to the consumptive use of water by native vegetation except as expressed in the general rainfall-run-off relation. The latter indicates that, generally speaking, from 40 to 100% of the rainfall is consumed by the cover. It goes without further explanation that the conservation of rainfall through the control of the consumptive use in the water-sheds would be of unestimable value.

To further such results, it is imperative that country-wide, systematic investigations be made of the consumptive use of the water-shed cover, including the water needs of individual plant species, the distribution of rainfall, the effect of over-year storage, and the multiplicity of other factors which affect penetration and run-off.

The idea is repeatedly advanced by discussers that rainfall, after it reaches the ground, remains evenly distributed, practically, and that absorption is uniform to the depth of plant roots, regardless of the ever-present irregularities of the surface of the ground and the lack of uniformity of the residual soils of the mountain water-sheds or of alluvial deposits in the foothill areas and steep valleys adjacent to mountain ranges. Hence, the discussers have repeatedly asserted that no deep penetration can occur unless the field capacity of the soil down to the root limit is first satisfied over the entire area under consideration.

Doubt has been expressed by some of the discussers as to the comparatively large quantities of deep penetration in the valley floor which the writer believes become available. In this connection, the study of water supply from closed basins should furnish conclusive evidence. Such studies are available for the San Bernardino Basin in the water-shed of the Santa Ana River, above Prado, in Southern California. Reference is here made to two independent investigations of the water supply of this basin, one by the Engineering Offices of J. B. Lippincott, M. Am. Soc. C. E., in Los Angeles, Calif.,† and one by the State Engineer's Office of California.‡

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* Cons. Engr., Los Angeles, Calif.

† Report on Water Conservation and Flood Control on the Santa Ana River for Orange County, July, 1925, Table 16.

‡ "Santa Ana River Investigation," *Bulletin No. 19*, State of California, Dept. of Public Works, Div. of Eng. and Irrig., 1929.

The water-shed of the Santa Ana River above Prado is a closed basin; it is bordered on all sides by mountain ranges embracing a valley area, which is a sunken basin many hundred feet in depth, filled with alluvial materials. This area is capable of absorbing a considerable percentage of water. A large portion of the tributary mountain water-shed enjoys a prolific rainfall and run-off. The latter has been measured by the U. S. Geological Survey in the Santa Ana River and numerous creeks, for a sufficient period of time to permit of accurate estimates, of total run-off reaching the valley. The principal stream is the Santa Ana River (water-shed, 203 sq. miles). The river, after traversing the basin and intercepting its tributaries, enters the gorge of Lower Santa Ana Canyon where a gauging station has been maintained near Prado since 1919-20. There is also available a large number of current-meter measurements at this point, made at more or less frequent intervals, since the early Nineties. The cross-sectional area of the underflow at this point is known. Surveys as to the use of water for domestic purposes and irrigation within the basin are available; surveys also are available of the moist areas from which water evaporates. Furthermore, there are rainfall records, covering long periods, of a number of stations in the valley floor and fluctuations of the water level in wells. In short, there are available comparatively complete data to establish the water supply, use, and waste in this closed basin.

In the Lippincott report previously mentioned, the possible sources of water supply, mountain water-sheds, foothill areas, and valley floor, were credited with the quantities listed in Table 19. These data are the averages for the 29-year period from 1895-96 to 1923-24.

TABLE 19.—TOTAL SEASONAL WATER CROP OF SANTA ANA RIVER ABOVE PRADO, CALIF.

Region.	Area, in square miles.	Water Crop, 29-Year Mean.	
		Total, in acre-feet.	Acre-feet per square mile.
Mountain water-shed, north and east	547.01	193 142	353.08
Temescal Creek	200.74	1 509	7.52
Foothill areas, north and northeast	68.26	22 328	327.10
Foothill areas, northwest	48.16	27 168	564.12
Foothill areas, south	40.73	2 748	67.46
Valley floor, north and northeast	245.88	46 644	189.70
Valley floor, Chino Creek	249.52	54 024	216.51
Valley floor, south	84.89	13 428	158.18
Total	1 485.19	360 991	243.
Santa Ana River, at Prado	....	153 102	....

The water crop attributed to the valley floor, 580 sq. miles in area, is nearly all deep penetration and amounts to 114 000 acre-ft. per annum, or more than 30% of that of the entire water-shed.

For the purpose of comparison with Table 19, estimates of rainfall penetration in the valley floor of San Bernardino for the seasons 1926-27 and 1927-28 are presented in Tables 20 and 21.* These statements are based upon penetration experiments and studies by Mr. Blaney, and discussed by him in connection with the paper.

The water supply characteristics of the two seasons are expressed in the index of wetness (percentage of long-period mean rainfall) and index of run-off (percentage of long-period mean run-off of the mountain streams), to wit:

	1926-27.	1927-28.
Index of wetness (City of San Bernardino)....	128	87
Run-off index	147	34

Tables 20 and 21 indicate that under certain conditions the water supply from rainfall penetration in the valley may assume substantial proportions and may become a factor of economic importance.

Mr. Blaney has contributed a valuable discussion and much useful data on evaporation and transpiration losses obtained in Southern California by the method of systematic soil moisture determinations. For areas with various types of soil, cover, and culture, this in many cases may be the only method available for the determination of the percentage of deep penetration. As stated in the paper, the method is subject to the errors resulting from the assumption that rainfall remains uniformly distributed after it reaches the ground and that there is no erratic variation of soil texture. The method is not so successful with very gravelly soils, because it is difficult to obtain samples except by digging shafts.

TABLE 20.—SAN BERNARDINO BASIN: SUMMARY OF ESTIMATES OF PENETRATION WITHIN VALLEY FLOOR FOR 1926-27 AND 1927-28.

Basin.	Area, in square miles.	Mean rainfall, in inches.	MEAN PENETRATION.		Total penetration, in acre-feet.
			Inches.	Acre-feet per square mile.	
Upper:					
1926-27.................	124.1	21.9	7.62	407	50 700
1927-28.................	124.1	17.6	1.68	86	10 700
Jurupa:					
1926-27.................	99.1	18.4	4.17	223	22 000
1927-28.................	99.1	16.1	.77	41	4 100
Cucamonga:					
1926-27.................	268.9	22.9	8.85	472	126 700
1927-28.................	268.9	15.2	2.93	156	41 700
Temescal:					
1926-27.................	55.9	16.0	3.02	161	9 000
1927-28.................	55.9	14.0	1.52	82	4 600
Lower (Coastal Plain):					
1926 27.................	305.6	16.1	4.87	260	79 200
1927-28.................	305.6	15.8	4.70	251	76 900

The extensive and thorough investigations now being carried on by the Division of Agricultural Engineering, U. S. Department of Agriculture, some of which are under the immediate direction of Mr. Blaney, will throw light on many phases concerning the problem of rainfall penetration. Comparison between penetration resulting from protracted storms and sprinkling experiments carried on in dry weather should lead to conclusions regarding the effects of atmospheric conditions, which are difficult of analysis.

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* *Bulletin No. 19*, State Dept. of Eng., California, Tables 22-*A* and 22-*B*.

TABLE 21.—SAN BERNARDINO BASIN: ESTIMATE OF MEAN PENETRATION VALUES WITHIN VALLEY FLOOR, IN ACRE-FEET PER SQUARE MILE, FOR VARYING MEAN RAINFALL.

Mean rainfall, in inches.	MEAN PENETRATION, IN ACRE-FEET PER SQUARE MILE.				
	Upper Basin.	Jurupa Basin.	Cucamonga Basin.	Temescal Basin.	Lower Basin (Coastal Plain)
9	0	0	0	0	0
12	0	0	27	5	91
15	0	0	150	123	224
18	118	187	272	250	360
21	336	336	390	390	490
24	500	470	520	520	620

Mr. Sopp's statement as to the results of soil sampling in a shaft 170 ft. deep is in confirmation of the writer's views and experience, and contradicts the theory advanced by many engineers that rainfall of the intensity characteristic of Southern California would not, and could not, penetrate a soil prism 170 ft. What most water supply engineers did not expect, however, and some still dispute, is that the percentage of rainfall reaching the water-table is as large as the writer's investigations have led him to conclude.

Mr. Troxell takes issue with the analysis of mountain run-off as shown in Fig. 8 and with the determination of the consumptive use of the cover from the "total run-off" curve. The values obtained by the writer's method represent averages over a period of years in which the effects of individual years are ironed out. For larger water-sheds with perennial flow, this method is probably the only one that is practicable, since it becomes difficult to trace the effect of individual years on account of the long-period storage which takes place. For a small water-shed like that of Devil Canyon, the following considerations will apply:

(*a*) For a dry year, preceded by a wet year, the curve gives values of consumptive use that are too large because of a draft on over-year storage (see year 1923, on Fig. 8);

(*b*) For a wet year, preceded by a dry year, the curve gives values of consumptive use that are too low, because a part of the rainfall remains in storage (see year 1922, on Fig. 8); and

(*c*) For the second and third years of a series of years of the same wetness, the curve gives correct values (see years 1920 and 1921 and years 1924 and 1925).

The larger the water-shed the more complex becomes the effect of hold-over storage. Considering further that, as a rule, data relative to the distribution of rainfall over large water-sheds are fragmentary, that no two seasonal rains fall in the same manner, and that moisture conditions and conditions of the cover of a water-shed are likely to vary from year to year, practical results can be obtained only by methods which iron out the irregularities of individual years.

Just why Mr. Troxell criticizes the storm run-off curve of Fig. 8 is not clear. The curve speaks for itself; it deviates very little from the construction points. The discusser states that "storm run-off depends not on seasonal rainfall, or even storm periods, but on the instantaneous rate of rainfall and the condition of the canyon storage". A determination of the storm run-off of a mountain water-shed from the instantaneous rate of rainfall is too impractical to deserve serious consideration. The method is used to estimate peak flows in urban areas, where the instantaneous rate of rainfall is known. In mountain water-sheds rainfall stations, as a rule, are few and far between, and daily or weekly rainfall records are the best to be expected.

Mr. Troxell's application of the run-off curves of Strawberry and Waterman Canyons to Lone Pine Creek is hardly fair. The first two water-sheds are short, steep slopes on the south side of the San Bernardino Mountains, exposed directly to the moisture-laden clouds drifting in from the ocean and receiving a very prolific rainfall. Lone Pine Creek, on the other hand, is on the lee or desert side of the highest peak of the Sierra Madre; its northerly slopes are formed by a low range of desert hills. The creek bed runs along the San Andreas fault rift for a distance of 10 miles; climatic conditions are those of a desert valley. No wonder the results of Mr. Troxell's comparison were disappointing.

Mr. Rowe's discussion divides itself into two distinct sections. The first refers to the consumptive use of water-sheds near San Bernardino, with which Mr. Rowe is very familiar. His data as to the effect of a denudation of the brush cover by fire are very instructive. The quantity of deep penetration from rainfall of 250 acre-ft. per sq. mile as determined by the writer for the brush-covered part of the San Bernardino area, is questioned by Mr. Rowe.

The second section of Mr. Rowe's discussion refers to the determination of rainfall penetration in Pauba Valley and on the Big Mesa of Temecula River. A casual trip over the Big Mesa is hardly enough to enable a person to become familiar with this water-shed or with the writer's investigations.

Relative to the penetration experiments on the Big Mesa, Mr. Rowe refers to the existence of hardpan at 4 ft. below the surface and states that this is the greatest depth to which rainfall ever penetrated. The writer made borings on the Big Mesa after the great storm of February, 1927. Some of the samples taken from approximately 16-ft. depths were wet and plastic. This moisture must have found some way to get below the so-called impervious layer of hardpan.

Mr. Rowe attributes special significance to the fact that on the Big Mesa there are two wells in which water stands higher by more than 40 ft. and 65 ft., respectively, than in adjoining wells. In reply, it may be stated that it is conceivable that a well would terminate in a thick clay stratum which is above the general water-table. Such a well, naturally, would drain the suspended water-table produced by the clay stratum in accordance with Fig. 1.

The well, therefore, would hold water, but its level would be above that of the general water-table.

Engineers interested in rainfall penetration are indebted to Mr. Lee for an instructive discussion which has brought to light many new and important points. The discussion relative to mountain run-off and "evaporation opportunity" is particularly useful.

Referring to the discussion of the computation of rainfall penetration from test wells in Pauba Valley, Mr. Lee's method of determining the factor to convert the observed rise of a water-table to the equivalent depth of water, is a decided improvement over the assumption of a flat percentage; but the elements which enter into such computations are more in detail than would ordinarily be available to the engineer in making a water-supply study, except for small areas. The writer's percentage of porosity of 25% was the result of the study of a number of well logs.

The following remarks relative to the Pauba Valley test wells are applicable to the discussions by both Messrs. Lee and Rowe.

Approximately sixty-four wells were used in making the study of rainfall penetration in the Pauba Valley. Practically all these wells were shallow; many of them penetrated only 2 or 3 ft. into the water-plane. Most of the wells were located in, or adjacent to, fields which were irrigated approximately once a month during the summer season. These wells within the irrigated area showed a rise following the application of irrigation water very similar to the rise which followed rainstorms. The deduction was made that if an application of irrigation water of 5 to 6 in., at a time when no storm run-off was present in the river, produced a rise in the wells, a similar rise would occur after a rainfall of 5 or 6-in. intensity to, roughly, the same amount, and it would be logical to conclude that this rise had been produced by the rainfall rather than by transmitted pressure.

Some of these irrigation rises, from May to September, are shown in Fig. 5, but they are much clearer in corresponding graphs of many of the other wells. It was impossible to present all the data upon which the deductions were based.

The conclusion was advanced by the discussers that transmission of pressure, rather than local rainfall, is responsible for the rise in the test wells. This is a question which may be of general interest. In explanation of any such theory, it should be stated that Pauba Valley is a closed basin; the river enters by way of Nigger Canyon Gorge and leaves by way of Temecula Gorge (see Fig. 4), with bed-rock outcropping in the river bed in both gorges. The gradient of the river averages about 38 ft. per mile. Below the upper gorge the river spreads over a wide outwash area, or débris cone, covered by deposits of sand and gravel. It is in this region that effective absorption occurs, because the entire summer flow, as a rule, seeps away. Pauba Valley is several hundred feet in depth and is filled with alluvial materials capable of absorbing about 25% of water as disclosed by the well logs. As the underflow approaches the lower gorge it is obstructed by the bed-rock formation and is forced to the surface, producing a rising stream which begins as a trickle about 6 miles above the lower gorge and increases to about 10 sec-ft. as the river approaches the gorge. The combined effect of the bed-rock and the resistance to percolation by the valley fill produces a pressure in the ground-water strata, responsible for the rising stream and for the flow in artesian wells. This is a phenomenon quite common with Southern California streams.

The water-table of the outwash area presents the fountain-head of this pressure. The process of ground-water movement, however, is believed to be a combination of pressure and percolation, the latter being the predominant factor for the reason that this basin, although only 9 miles in length between gorges, has the effect of long-period regulation of stream flow, with a summer flow varying from 8.5 to 12.5 sec-ft. In 1900, after a period of seven predominantly dry years, there was still a perennial flow at the lower gorge of about 10 sec-ft. during the summer months.

If the rise in the test wells was due to transmission of pressure, it would have had to occur either simultaneously with, or after, the appearance of the flood flow in the stream. As a matter of fact, a flood would naturally lag behind the downpour. It should also be noted that the flood channel in the outwash area may occupy only a fraction of the total width of the latter, unless the flood were to assume extraordinary proportions; that under ordinary flood conditions a certain lateral percolation must take place in the outwash area; and that a rise of the water-table over a considerable region before a transmission of pressure would affect the entire valley below. The lateral percolation observed, occurred under an oblique angle to the axis of the stream bed, and at rates varying from 16 to 33 ft. per day.

On certain of the test wells continuous records were kept which indicated that the rise preceded the flood flow at the outwash area. Many of the graphs also show that the test wells reached a peak before the wells in the outwash area above them and which, of necessity, would be the origin of any transmitted pressure. In most cases, after the occurrence of these peaks the test wells declined rapidly, while the wells in the outwash area were still approaching their high point. It would be logical to conclude that if the fluctuations in the test wells were dependent upon transmitted pressure, both the rise and the decline would follow the corresponding rise and decline of the levels in the wells in the outwash area rather than precede it.

Any conclusion that the rise in a test well might be due to a rise in the river opposite that well cannot be reached because the elevation of the water-table was higher than the river. It also appears that both Mr. Lee and Mr. Rowe expect a uniform manifestation of rainfall percolation, regardless of the unevenness of the ground and lack of uniformity of soil formation. Collection of rain water in depressions may readily account for a rise in some test well, greater than the rainfall would warrant.

Relative to the matter of rainfall penetration on the Big Mesa, which is doubted by Mr. Lee and Mr. Rowe, the question resolves itself ultimately as to the existence of a general water-table beneath the Mesa. A pumping test made from May 18 to June 15, 1928, on Well No. 131, located on the Mesa about 1½ miles east of Temecula (after the paper had been written), tends to support the writer's conclusions. The well is 12 in. in diameter and

604 ft. in depth. Its log shows 61 ft. of aquifer, 18 ft. of undetermined water-bearing capacity, and 525 ft. of impervious strata. The general features of the test are shown in Table 22.

TABLE 22.—PUMPING TEST AT WELL NO. 131.

Date.	Time.	Depth to water, in feet.	Output, in second-feet.	Remarks.
May 18, 1928	11:00 A. M.	63.04	......	......
May 23, 1928	10:00 A. M.	62.70	......	......
May 23, 1928	11:00 A. M.	82.40	0.841	Pumping at intervals; many interruptions.
May 24, 1928	7:00 A. M.	83.77	0.351	
May 25, 1928	7:00 A. M.	85.65	0.382	
		Changing casing		
May 25, 1928	9:00 P. M.	111.58	0.789	......
May 29, 1928	12:00 M.	105.10	0.837	Constant pumping for 29 hours 40 min.
May 30, 1928	5:40 P. M.	115.70	0.799	Stopped pumping
May 30, 1928	5:48 P. M.	78.69	......	Not pumping.
May 30, 1928	6:40 P. M.	71.63	......	Not pumping.
May 30, 1928	7:40 P. M.	70.49	......	Not pumping.
May 31, 1928	7:00 A. M.	67.66	......	Not pumping.
June 1, 1928	7:10 A. M.	66.10	......	Not pumping.
June 10, 1928	9:35 A. M.	63.70	......	Not pumping.
June 15, 1928	4:00 P. M.	63.41	......	Not pumping.

The total quantity of water pumped was 3.63 acre-ft. The volume of water pumped and the rate of recovery of the water-shed would seem to warrant the conclusion that a general water-table underlies the Big Mesa.

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Paper No. 1757

THE PLANNING OF CAPITAL CITIES: DENVER, COLORADO*

By S. R. DeBoer,† Esq.

With Discussion by Messrs. Morris Knowles, Richard R. Lyman, William T. Lyle, George H. Herrold, and S. R. DeBoer.

Synopsis

As this is the first of an intended series of papers on the "Planning of Capital Cities", some general remarks may serve to define their scope.

Capital cities dot the map of the United States from ocean to ocean and from Mexico to Canada. They include varying conditions of climate, commerce, and physical characteristics; but they are alike in that each is the seat of government of a Sovereign State. Some are old, others are relatively new and are still developing "growing pains". Some are located on ocean, lake, or river, some enjoy the clear vision of mountain slope or range. A few are located on outstretching plains, and many are favored with an eminence on which to place their significant building which is the emblem of State authority.

It is always of interest in studying the origin and development of an important city to know who made the first street plan; how this has been enlarged from time to time; and whether the city has a planning commission and has been zoned. Because of the many lessons that can be drawn, and the older history of development which can be studied, reference is made in the paper to several of the better known European capitals. The City Planning Engineer can find much of interest to study in foreign capitals.

After noting the general characteristics of capital cities, the paper presents a description of Denver, the capital of Colorado, and outlines its early development and transitions in stages of growth. Attention is called to the various

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attempts in city planning; to the fact that the location of the Capitol has acted as a nucleus and, to some extent, has dominated the street system.

Considerable attention is given to the recreational system of Denver and its environs, as well as to the application of principles in the development of the Denver plan, and the unique features of zoning which are applied thereto.

General Characteristics of Capital Cities

A city may be termed the "service station" of its district. It performs the work which the surrounding region requires. Thus, based on their means of existence, cities can be classified as factory, mining, agricultural, commercial, residential, or tourist cities; health resorts or educational centers; and military or capital—that is, governmental—cities. It is true that most urban communities include more than one of these classifications, and large cities often cover all of them; but in the life of each, except the metropolitan center, some one feature is predominant. In the United States, few capital cities have a metropolitan character. In planning for their future, it is proper, therefore, that major emphasis be placed on those features which will focus attention on the seat of the Government.

It should be remembered that, not only are the reins of government centered in a capital city, but also the pride of a State or of a Nation. Modern civilization is not as yet far enough removed from the feudal days, but that the pomp and splendor attending governmental functions are still objects of admiration. This interest, however, is shifting from individuals to buildings and cities. Democracy has taken the diamond-studded uniform and sword from the rulers of this country, but its citizens still want to point with pride to their Government, and this pride is more and more becoming embodied in buildings and cities.

These fundamental facts determine the character of the plan according to which a government city should be built. It must be laid out on liberal lines, with wide boulevards and attractive parks. It must have large, open plazas, around which imposing governmental buildings can be grouped, and the main group with its plaza should be developed into a civic center.

Foreign Capitals

The civic-center idea reaches far back into the history of Rome when the Emperors beautified large areas for the city, mostly for their own glorification; but the idea of beautifying capital cities and planning them from the outset had its origin with Louis XIV, the self-styled "King of the Sun", of France. He hired Le Notre to plan the gardens and city of Versailles on a scale of magnificence which the world, at that date, had not witnessed. Jealous of his fame, other princes began to build similar cities, or to beautify their existing ones. A German prince immediately began the building of Karlsruhe. Czar Peter the First built St. Petersburg, and then other cities—Vienna, Berlin, and Paris—adopted extensive improvements.

The City of Paris had corrective and beautifying plans as far back as 1664. It seems that the Royal House of France retarded the development of these plans, fearing the City would become too powerful. The first basic plan was developed after the Revolution, and it was partly carried out by the First Napoleon as a military necessity in controlling the city. The Third Napoleon, with the assistance of Haussmann, completed more of the work, also for military purposes. In spite of the military influence, Paris has developed a plan that is following the general lines characteristic of a capital city, with the exception of the location of railroad stations, which have no particular relation to the center of the city.

The City of Vienna is an interesting illustration of a capital developed around an existing old city. The "Ring Street", built on the old fortifications, has become the location of the Government buildings and the royal palaces. The outer streets radiate from this circle boulevard. The inner part of Berlin is carefully planned with radiating traffic lines and circular boulevards, which are placed mainly on the old fortifications. The newer parts of Berlin are not as well planned as the older part.

The City of London has no apparent design. It has been built on the principle of expediency of the moment rather than by following a comprehensive idea. In spite of this, the Houses of Parliament, Buckingham Palace, and the Administrative Offices along Whitehall are more or less connected. The remainder of the city seems to follow the lines of cowpaths rather than planned city streets. Through the conglomeration of blocks, major streets wind their more or less indirect ways. The proposal of the great Christopher Wren would have made London a well-planned city after the fire in 1666; but it was never adopted. The reason London can exist and grow, in spite of this street handicap, is the fact that its underground transit system supplies the need for arterial and circular lines.

Karlsruhe, Germany, represents a good illustration of a city built along radiating lines. All streets are focused on the governmental center—in this case, the palace of the ruling prince. Two-thirds of the circle is occupied by the Royal Park placed back of the palace. The city in front is well planned, with open plazas in various places. Triangular plots, caused by the intersection of radiating lines, are carefully avoided.

All the plans thus far discussed were designed before modern means of transportation affected the city plan. The plan of Canberra, Capital City of Australia, takes into consideration not only this, but also the still newer ideas of zoning. Each of several centers is laid out as an independent city, connected with the other units by broad and direct lines of traffic. Canberra has a main civic center—the group of Government Buildings. Around it are grouped secondary centers, such as the municipal, business, residential, industrial, and the suburban districts.

THE CAPITAL OF COLORADO

All these cities are capitals of large nations, in which the Government center is dominant. In the forty-eight State capital cities of this country, the governmental feature is of less consequence. Many of them, however, are typically administrative cities, and would not be what they are to-day were it not for this activity.

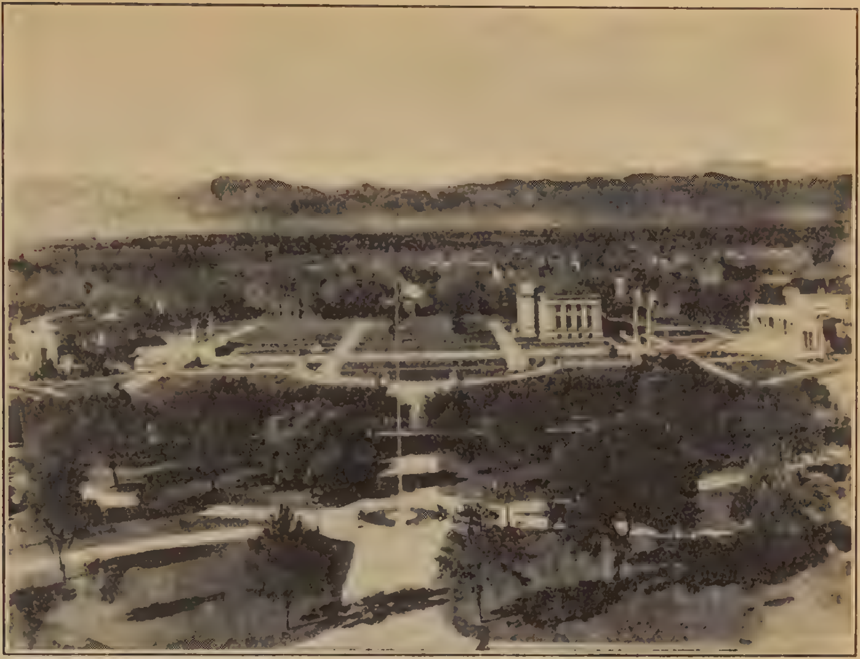

FIG. 1.—DENVER CIVIC CENTER, WITH MOUNTAIN RANGE IN BACKGROUND. VIEW FROM CAPITOL, SHOWING OPEN-AIR THEATRE AT LEFT, LIBRARY IN CENTER, AND COLONNADE AT RIGHT.

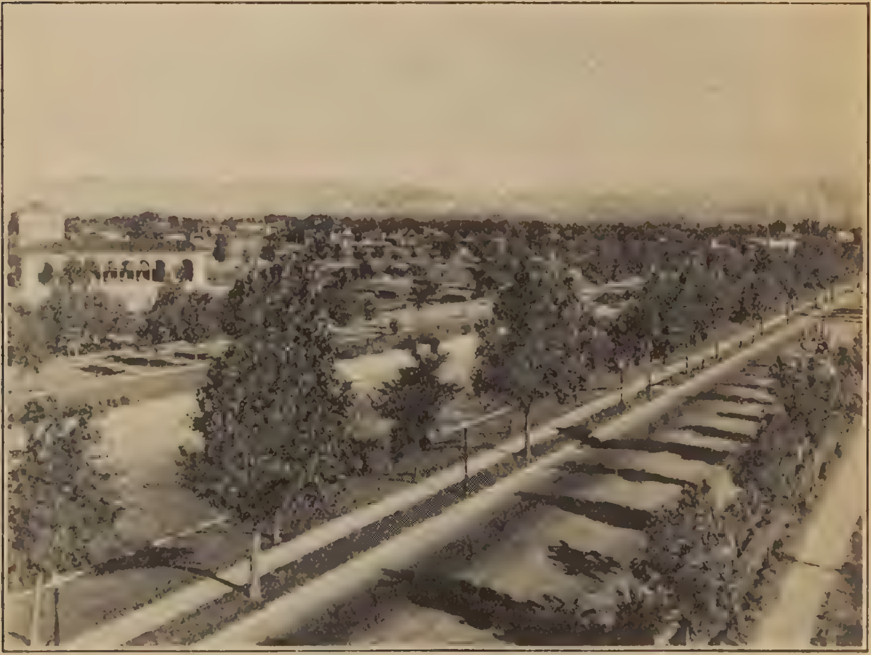

FIG. 2.—VIEW OF SPEER BOULEVARD, DENVER, COLO.

The City of Denver is an outstanding example of a capital city in America. It is third in population among State capital cities. Built at the foot of the Rockies (Fig. 1), it has as its background 200 miles of the Rocky Mountain range, towering up to peaks more than 14 000 ft. high. As seen from the city, this affords one of the most magnificent vistas any capital ever had, but, at the same time, it has been one of the most serious barriers to commerce. Denver had its beginning as a mining town in 1858, and immediately became the leading city of a large district. Many of the mountain valleys open into the plains near the city, and it soon became the distributing point to the mining cities in the mountains.

When the first transcontinental railroad was built by the Union Pacific Company, the proposed line over Berthoud Pass was abandoned as impractical, and the line was built through Southern Wyoming, with Cheyenne as a division point. It was a hard blow to the small city of Denver, and it is said that its population decreased to one-half. Many cities are aided rather than harmed by serious calamities; the fight and co-operation necessary to overcome them may carry the city to far greater success than it would have reached otherwise. The citizens of Denver were jarred into action by this location of a railroad that ignored their city. They built a connecting line from Denver to Cheyenne. When they realized that they actually could do such things, it took but a few years for them to build other lines, and, thus, at the time Colorado was admitted to Statehood, the city was the center of eight railroads.

The Rocky Mountains have always been a barrier against the city's railroad connections to the West. Denver has thirteen railroad lines reaching out in all directions, with three minor lines reaching into the mountain region. In 1907 a railroad was built over the mountain range, with a maximum altitude of nearly 11 000 ft. This road has wonderful scenic advantages, but transportation during the winter is almost impossible. To avoid the heavy grade over the range, the Moffat Tunnel six miles in length has been built through the mountains. It was financed by the counties served, Denver carrying the major part. This tunnel will eventually put Denver on a major transcontinental line. Connecting lines west of the tunnel are still to be built, but once this main line is in operation, the distance to the Pacific Coast and to Salt Lake City, Utah, will be reduced by 250 miles.

Being located in the arid region, its water supply is another problem of the City of Denver. It has ample water for domestic use at present, but, with the expected growth, the city is laying plans for a far greater supply. The quantity of water contained in the drainage areas of the State is limited, and a large part of it must be used for the irrigation of farm lands. The city cannot utilize this irrigation water because, with the growth of the municipality, the watering of a larger acreage of farm land around the city will become as essential as the greater domestic supply. Scientific use of water will be one of the problems the city must face. Plans have been effected to use the pioneer bore of the Moffat Tunnel for an additional supply of water to be carried from the western slope of the mountains to within reach of the city.

A bluff overlooking the city was selected for the site of the State Capitol. Three blocks of ground were acquired, sloping up from the city's main thoroughfare to the top of this hill on which the Capitol was built. The building was placed in such a way that it commands a beautiful view of the Rockies (Fig. 1). The tower is located on the axis of the city's main shopping street, but the building is shut off from this street by one block of stores. In the early days, the city failed to open this shopping street to the Capitol grounds, and since that time such efforts have been unsuccessful.

Fifteen years after the completion of the State Capitol, the city prepared plans for the development of a civic center, as a continuation of the Capitol grounds, and on the same axis. Two and one-half blocks of ground were acquired, on which the Library, an open-air theatre, a fountain, and an ornamental colonnade now stand (Fig. 1). In 1924, the city acquired another block for the erection of a new City Hall and Court House. Denver is fortunate in having a definite civic square on which, in the future, all its major public buildings can be made to face. Tentative plans for the continuation of this State and Civic Center have been made, and there is no doubt but that, eventually, this park will be more than $\frac{3}{4}$ mile in length. The Federal Mint is already located on this extension, and an art gallery will very likely follow.

Denver is still an infant among cities. It is scarcely more than threescore years old, but it may be stated that it has sensed its calling as a capital better than many an historical city. At a considerable expense, it has acquired this civic center in front of the State Capitol, and has begun laying it out in one of the best known, if not the most attractive, municipal centers of the world. This has proved to be a good investment. The actual return is an indirect one, and is difficult to estimate, but it is said that the city's added income pays for the cost of its Civic Center every two years.

Regardless of the accuracy of these figures, the value of this Civic Center is shown in the attitude of the citizens themselves. From active opposition during the period of construction, this viewpoint has changed to one of impatience for any delays in continuing the work, and of anxiously guarding the treasure against mistakes of design or vandalism.

Denver is planning another Government center. The Federal Building and Post Office was in a remote corner of the commercial section. For many years the location has seemed unfortunate, from a business standpoint and because it does not tie in to the State and municipal group. There was a time when the Civic Center needed this extra building. Time, however, healed this, also. Down-town Denver is growing to its Post Office, and the added confidence in its Civic Center has made the Federal Building unnecessary to the municipal and State group. Other Federal Buildings are needed, however, and now the City contemplates a special plaza for this purpose. The people of Denver would not think of a second center had its main Civic Center not been a success.

The population curve of Denver shows a steady growth of 2.1% annually. At the present rate, the city will have a population of 520 000 in 1950, and plans are being completed for a major street layout based on this estimate. Traffic studies in the down-town district have shown conclusively the need of a belt boulevard for traffic around the business district. This line will consist of wide boulevards that will tie the municipal and State centers to the Government center, to the hotel district, to the Union Station, and to the major arterial lines. It will encircle the business district, and traffic can enter from all four sides. This belt boulevard will take on very much the character of the Ringstrasse of Vienna or Cologne and, when completed, will be monumental in appearance. From it radiate all the arteries of the city.

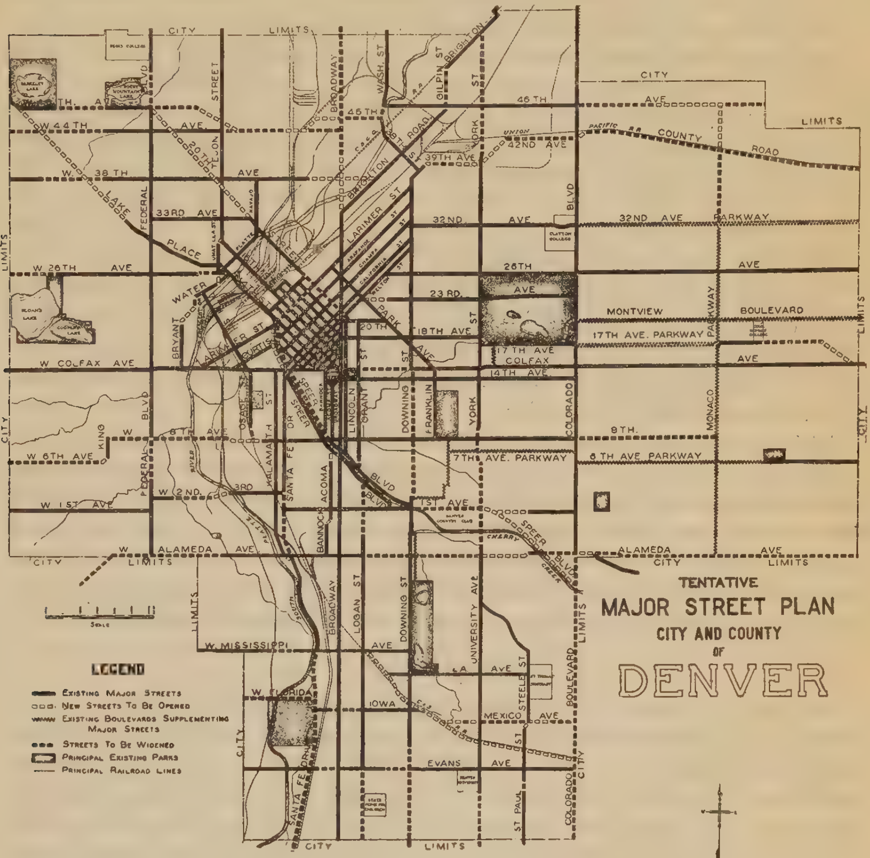
TENTATIVE
MAJOR STREET PLAN
CITY AND COUNTY
OF
DENVER
LEGEND
Existing Major Streets
New Streets To Be Opened
Existing Boulevards Supplementing Major Streets
Streets To Be Widened
Principal Existing Parks
Principal Railroad Lines
Scale
CITY LIMITS
W 44TH AVE
W. 38 TH AVE
33RD AVE
W 26TH AVE
W COLFAX AVE
W 6TH AVE
W 1ST AVE
W. ALAMEDA AVE
W. MISSISSIPPI AVE
W. FLORIDA
IOWA
EVANS AVE
MEXICO AVE
46 TH
42ND AVE
39TH AVE
32ND AVE
26TH AVE
23 RD.
20 TH
18TH AVE
17 TH AVE
COLFAX
14TH AVE
8TH.
7TH AVE. PARKWAY
8 TH AVE PARKWAY
1ST AVE
ALAMEDA AVE
MONTVIEW BOULEVARD
17TH AVE. PARKWAY
32ND AVE PARKWAY
PACIFIC COUNTY ROAD
LAKE PLACE
FEDERAL BLVD
BRYANT
WATER
BROADWAY
WASH. ST
GILPIN ST
BRIGHTON
YORK ST
LARIMER ST
PARK AVE
SPEER BLVD
CHERRY CREEK
SANTA FE
KALAMATH ST
ACOMA
BANNOCK
LINCOLN
GRANT
DOWNING ST
FRANKLIN
YORK
COLORADO BLVD
MONACO PARKWAY
UNIVERSITY AVE
STEELE ST
ST PAUL ST
LOGAN ST
SOUTH PLATTE

FIG. 3.—DENVER'S MAJOR STREET PLAN—STATUS AS OF DECEMBER, 1927.

The main radial line of the city is Speer Boulevard (see Fig. 2), built on the borders of Cherry Creek. At the time of its construction, the creek bottoms were the dumping grounds for the city. The Boulevard was much opposed, but its value is now shown in the 26 000 cars that pass over it daily. As an attractive feature in the Denver plan, it is just as important. The continuation of the Boulevard north and south is one of the proposed lines of the present plan of Denver (see Fig. 3). The other major traffic boulevard that will become part of the belt is the present 20th Street. Its widening and extension will carry it to the northern entrance of the city, and south to a line that reaches to the southern limits.

Broadway is one of the major traffic streets of Denver. It was extended in 1919, and it is now (1929) contemplated to continue it to the city limits north. Many other radial lines are being considered, some of which will necessitate the building of viaducts over the Platte River. The main belt boulevard will be followed by other lines that reach out in large ellipses around the city. One of the boulevards follows the railroad lines, and ties together in the eastern part of the city on an existing parkway. This circular boulevard, in its down-town part, becomes a by-pass road for an east and west line.

Denver's recreational system includes approximately 1 664 acres of city parks and 11 006 acres of mountain park land. The City tries to maintain its parks in the highest degree of beauty and efficiency. Numerous roadways have been eliminated from the larger parks, to save them from becoming short-cut routes. The traffic has been collected on arteries bordering the parks. The policy of using these border streets as major arteries of traffic has the advantage of creating unbroken traffic lines, and gives the motorist some benefit of the park without allowing him to destroy its quiet atmosphere.

Plans for a systematic playground development are under consideration. If these are adopted, the park and school playgrounds, combined, will furnish one playground for every ¼-mile radius. The school system of Denver is also being developed by means of carefully studied plans. A school is now provided for every ½-mile radius of the city.

In its zoning plan, Denver has adopted the principle of large areas for one-family homes. The city is built for light and air. Ground is cheap, and any one can afford to build on sufficient area. Height regulations existed in Denver before zoning ordinances were developed, with the result that the city has a uniform twelve-story skyline, harmonious with the idea of a capital city. The industrial district is located along the railroads, running in a narrow strip from the extreme southern limits of the city to the northern boundary.

Conclusions

In the development of the Denver Plan, there has been opportunity to determine just how far general planning principles, derived from the study of classic examples in European capitals, can be made to apply to American conditions. It is also interesting to determine, from the study of specific cases, what the broad fundamental principles of a problem seem to be, and to what extent they will be useful for general application. From experience at Denver and general studies made in conjunction therewith, it appears that the theoretical plan of a capital city can be divided into four major zones: The zone of Government buildings; the central business district; the residential zone; and industrial zones.

The Zone of Government Buildings.—This is an area not usually found on city zoning plans. The City of Washington, D. C., has this zone, divided into two focal points—the Houses of Congress in one, the Executive Buildings in the other. For a smaller State capital, it will not be possible to plan on such a large scale. The State Governmental group will usually consist of Capitol buildings, additional office buildings, Historical Museums, State Armory, and often a Federal Building. These buildings should be grouped around a large and attractive square. A second open square should contain all the buildings serving the municipality, including the City Hall, City Auditorium, Post Office, Library, Chamber of Commerce Building, etc. This second square should be also well designed and proportioned, but may be slightly less imposing than the former.

The Central Business District.—The two squares should be directly connected by one, two, or three broad shopping streets. In the planning of this shopping zone, the governmental character of the city must be kept in mind. The people of a Commonwealth want to be proud of their State House, of the grounds around it, and of the city surrounding all. Thus, the shopping zone must answer this requirement. Streets should be wide and building heights should be well proportioned to the width. The building height limitation of the office buildings must be such as to avoid dwarfing the Capitol buildings.

The Residential Zone.—The residential districts should be built on similar lines. Here, more than anywhere else, must the capital city outshine every other city in the State. It must have attractive residential streets, ornamented by unusual horticultural treatment, with broad parkways, and curving roadways along creek and river beds. The apartment house zones should be placed around open squares of park. The residential district should be so inviting as to give every one of the State's citizens the desire to move to the capital city eventually.

Industrial Zones.—These should be carefully located away from the other zones, and should be on the outer border of the city. They can be made large enough to sustain a fairly large city. The residential areas essential to them should be placed between the industrial zone and the governmental area.

With the governmental square and the civic center located, the railroad station should be the third point of a triangle. The three features should be connected by direct and wide street lines and, possibly, by a boulevard running in a circle around the city's shopping center and connecting these three features. The station square must be attractive and set off by hotel buildings.

From each of these three center points, major traffic streets should run in all directions. Every point should have at least six different street lines radiating from it, and centering on the important building of its group—the State House, the City Hall, and the Railroad Station. The radiating street lines should be tied together by a circular boulevard surrounding the whole city.

This theoretical plan is naturally subject to the topography of the site. In case of the governmental group, it may be possible to select a high promontory, as was the case in Salt Lake City, where the State House towers over the Utah plains as the Acropolis once dominated Ancient Greece. Such an arrangement lends itself to great beauty, and a city should not fail to make use of it. The city center must bear a close relation to the shopping district, but there is no reason—if it were possible—why this square might not be on a second and lower hill, leaving the shopping center in between in a slightly lower area.

The time for low and relatively small governmental buildings will soon be passed. Capital cities must plan along bigger lines than they have dared to dream of thus far. The country has not reached its maximum growth, or population, by any means. At the rate of density of the population of present-day Germany, this country may eventually hold a population of 800 000 000 people. This means a sixfold increase, which is not only possible, but very probable. It will mean a proportionate growth in the cities, especially in the capital cities.

Denver is absolutely self-conscious in regard to its needs and its destiny. To be the capital city of a proud Commonwealth, it must be attractive for visitors; it must be good enough to live in; it must be clean and hospitable; and, even more, it must have the imposing character that the people can idolize.

DISCUSSION

MORRIS KNOWLES,* M. AM. SOC. C. E.—The City of Denver is one of the newer capitals of the West—one that has many rare combinations of outdoor expanse, scenery, and other attractions, the value of which is so well recognized that plans are being made not only for the city interior, but for all the surroundings.

The author properly calls attention to the fact that most cities of reasonable size, even if characterized by one particular line of activity, are, to a certain extent, cosmopolitan in nature. Few cities, like few persons, can well succeed in, nor are they destined for, a single-track existence. There are not many capital cities that claim to be devoted wholly to Governmental functions. Even in the National Capital, a well organized committee is having a survey made to discover whether or not more business can be attracted to the Federal City; and, also whether perhaps industry of a light, unobjectionable nature cannot well be conducted—not too near Governmental buildings.

Is it not true, then, that all cities, even capitals, must have a threefold purpose: (1) To provide homes—places in which to live; (2) to provide means of livelihood—facilities for working, doing, creating; and (3), finally, to provide for education, enjoyment, and recreation?

It may be worth while to pause for just a moment on the second stated object of cities, because there has been some change of feeling in the United States in regard to the strenuous competition between cities; there is some doubt as to the wisdom of the chambers of commerce, and of the people, in striving to enlarge and increase their cities, as if size alone were the desired object.

Much is heard as to how the city shall be made greater industrially or more important as a business market. There is discussion of a new activity called the "industrial survey"; an important and desirable review. Frequently, however, such survey has been confined to factories, types of industries (good and bad), market conditions, transportation conditions, etc. In consequence, it sometimes has become so concentrated upon these subjects of industrial supremacy, important as they are, that there has been a complete neglect of reckoning of the amenities, vital statistics, social values, community health, educational opportunities, governmental operations and effect of taxation system, the closeness of adherence to a good city plan and zoning; recreational opportunities, etc.

Thus, there has dawned on some a feeling that such communities are becoming unbalanced. That they are forgetting the complete development of which the author has written, and are devoting themselves to one thing. The Chamber of Commerce of the United States has called attention to the fact that there is danger in this kind of ambition and in this kind of survey when it attempts only to picture one's community as an aggressive, wise, thorough organization, seeking for industry and nothing further.

Study has been given, by the Civic Development Department of the Chamber, to the subject of what has been called a "Master Survey" Such a study would review the community in a large and comprehensive way and would consider all municipal functions. After thus learning about itself and what it must do to make itself attractive to industry—and to the people and their families who will engage in such pursuits—the city will be ready for the more detailed analysis of what it should do to care for the manufacturing and the business which it hopes to secure.

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The author has called attention to the subject of utilities. One often forgets that public utility planning is city planning. The city engineer may be said to have been the original city planner of America, because if he were "on the job", he never did anything without considering, as far as was humanly possible, what the future was going to be for that particular function, and how it was going to affect the growth and development of the city. As the author has wisely stated, one of the most important things for a city, particularly for a capital city, if it desires to proclaim itself as an attraction for tourists, is the water supply. This, in fact, is true of all forms of public utility service, but the Eastern cities never have been so seriously concerned with the question of water supply as those in the West because seemingly, but not absolutely, they are affected by less arid conditions. Nevertheless, their very prodigality has brought them trouble. In New York, N. Y., and Boston, Mass., for instance, it is becoming necessary to go farther and farther away for new supplies, because near-by sources have either become contaminated or polluted, or industrial wastes have made their use undesirable.

There is a Biblical truth which may be applied in such cases: "What is a man profited, if he shall gain the whole world, and lose his own soul?" To paraphrase, it might be said concerning some Eastern cities, "What does it profit a city to plan for wonderful industrial growth and allow the streams to become so polluted that the people working in the industries have no suitable drinking water?" And, conversely, "Where is the benefit if some large city of the arid West must provide its own needs by taking life itself from the very crops which are themselves needed for sustenance for the people of the city?" These things are by no means minor or insignificant phases of city planning. Cities must give way to the least important in all these problems if they are to have the well-rounded development necessary for people to live and exist as they should.

Mention has been made of the planning of Washington, D. C., and the creation, after some years of discussion and controversy, of the Capital Park and Planning Commission. This promises a new study and a new development commensurate with the dignity of the National Capital. In passing, it may be well to call attention to the rather important part which the Society has taken in the counsel and discussion, and in the decision, for wise legislation and the appointment of civilians to aid those in the official life of Washington, in providing that kind of planning which will insure a National Capital of which the whole country will be proud.

Recently, some interesting historical data have been brought to life in Western Pennsylvania, concerning the planning of what was intended to be a model community on the bank of the Ohio River, about eighteen miles below Pittsburgh. The plan itself antedates by several years the L'Enfant plan of

Washington. For information concerning this notable early bit of planning, credit is due to the Historical Association that has carefully preserved the records of what was known as the Harmony Society of Economites, a German religious sect whose followers came to this country in the early part of the Nineteenth Century. First, they settled at Harmony, Pa., on the Conoquenessing Creek. Later, they moved to New Harmony, Ind., an American hobby of Owen, the English mill owner. Finally, they abandoned this city because of an epidemic of malaria and, in 1824, moved to Economy, now Ambridge, Pa.

They were a frugal, careful, substantial people; they gained much of the world's goods and generally were considered to have a far-seeing wisdom. What is not generally known, however, is that they settled upon a site already laid out, which they bought from the heirs of one, Col. Isaac Melcher.

In 1786, Colonel Melcher and his associates secured from the Colonial Office a grant of 1 000 acres and laid out what he and his friends thought would be the capital city of Westmoreland County, which at that time was practically all of Western Pennsylvania. Some streets were laid out to be 99 ft. wide and others, 66 ft. The locations of the important buildings and churches were indicated. The town lots were 44 ft. wide and 221 ft. deep. A square of ample size was provided for a large public building and this site, later, was utilized by the Economites, for what they called "the great meeting house". This structure still exists. The land now is under the control of a Board of Trustees appointed for the preservation of the early town and building. The old plan follows the rectangular plan originated by William Penn, and shows an early appreciation of the design of a capital city. This example truly represents that old-time phrase that "there is nothing new under the sun."

Richard R. Lyman,* M. Am. Soc. C. E.—This paper on the planning of Denver, Colo., prompts the speaker to add his thoughts, especially with respect to street numbering. In Salt Lake City, Utah, the streets extend north and south, east and west. Most of them are 8 rods (132 ft.) wide.

For one who has been brought up on this system to be required to find his way in a city with streets so named that they do not indicate their location or their relation to one another, is an exasperating experience. People endure these conditions without complaint chiefly because they are accustomed to them, and do not really comprehend the great inconvenience.

Beginning at the Temple Block, the streets in Salt Lake City are numbered and named as follows: First South Street, Second South Street, Third South Street, etc.; First North Street, Second North Street, Third North Street, etc.; First East Street, Second East Street, Third East Street, etc.; and, First West Street, Second West Street, Third West Street, etc.

The houses are numbered east and west from Main Street and north and south from South Temple Street. Thus, there is no limit to the area over which such a system of naming and numbering streets and houses can be carried.

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To each block there are one hundred numbers; therefore, the house number and the name of the street give at once the location of the house and its distance from the southeast corner of the Temple Block—the initial point. For example, No. 728 East on Seventh South Street is located 7 blocks south from the place of beginning and 7.28 blocks east of East Temple Street. After examining the maps of several American cities, it seems to the speaker that the streets can be given numbers, and that the numbers can run north and south and also east and west from the two streets selected as the initial streets.

A stranger needs no map of Salt Lake City. In a few moments he can learn how to find his way to any address.

Some have objected to this method of numbering streets and houses on the ground that it would be like putting a man in a filing case with a number on him. Is not that the best method of putting away anything that is to be found again?

For purely business reasons it will be to the credit and advantage of any city to have its streets and houses so numbered that any one in any city can find easily the place he is seeking. The speaker has examined the maps of numerous cities with the thought of solving this problem in this way, and he is convinced that such re-numbering would be a comparatively simple matter.

William T. Lyle,* M. Am. Soc. C. E. (by letter).—The selection of "The Queen of the Plains" as the first of American capital cities to receive the attention of the Society is an appropriate one. Denver stands unique in many respects and should embody in external characteristics the civic consciousness of an extensive region.

The engineer in his conception of city planning is apt to drift into materialism. In the design of capital cities, even more than in others, the planner must avoid an emphasis which might be characterized as materialistic. Of course, the city must be a comfortable and healthy place in which to live, with every consideration given to those matters which facilitate trade, promote industry, and simplify transportation; but it must be much more than this. It must be a concrete expression of noble civic ideals.

Denver has unquestionably sensed several of these ideals and is bringing them out in the new city plan. The capital of Colorado is fortunately situated, as those who have seen the Rockies from the Capitol Building and have viewed from Inspiration Point, a glorious mountain panorama extending from Long's Peak in the north to Pike's Peak in the distant south, can testify. In addition to parks, boulevards, and a civic center within the city limits, advantage has been taken of splendid opportunities for the acquisition and development of mountain parks. In her civic center the majesty of the law and the universal dispensation of justice are held up to view.

There are other ideas and ideals that should be emphasized, some or all of which may be in the minds of the planners. Denver eventually will become an historic center and should perpetuate in visible expression the early life of what was at one time a frontier, and, later, became a great mining center in the rush for gold.

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The writer would suggest an emphasis on humanitarianism, charity, and morality without which civilization cannot survive. It would seem proper, especially in a city which has featured moral reform among the young, to bring out the idea of civic righteousness. Instead of zoning eleemosynary institutions practically off the map, they should be featured properly in the city plan. In medieval times, before the separation of Church and State, the Dom was featured prominently and was often the center of an attractive "street picture". To-day, the location of the city's churches should receive due consideration by the city planner in order to emphasize the religious idea in the life of the American people.

GEORGE H. HERROLD,* M. AM. SOC. C. E. (by letter).—The planning of a capital city requires a different conception of functions than that of the ordinary commercial or manufacturing city, although the plan must provide for those necessary adjuncts to city life—banking, retail business, industry, transportation, and housing. In considering a capital city there must be a clear thought of its political character. It is the State's city; the seat of Government; the home of certain elected officials representing the Commonwealth; and the people of the State have an interest centered in this city that is dominant and must be recognized.

The State Capitol Building is and should be of fine architecture. Its satellite office buildings for housing the various State activities should be placed to form an architectural grouping, a State Civic Center, noted for the nobility of its structures. Here is located the Governor, the State Supreme Court, and certain Constitutional officers, such as the Secretary of State, State Treasurer, and State Auditor. The members of the Legislature (and these members are of no small number) must be properly housed during the Legislative session, which may be for one-third of the year. All these people must be provided with offices of a type and character consistent with the dignity of a great State.

In addition to the business of Constitutional officers, there are many activities carried on by the State Government and under State supervision which are not found in any other city. In general, there are the following departments which require buildings to house officials, their technical and clerical forces and equipment: The State Automobile Department; the State Department of Agriculture; the Bacteriological Laboratories of the State Department of Health; the Adjutant General's Department; the State Highway Department with its locating, construction, and maintenance engineering departments, drafting rooms, testing laboratory, and purchasing department; the Department of Conservation; Drainage Department; Geological Survey; Forestry Department; the State Dairy and Food Department, with chemical laboratories; the State Department of Education; the Grain Inspection Department; the State Department of Health, Division of Preventable Diseases; and the State Department of Health, Division of Sanitation, including the engineering forces for inspection and supervision of construction of sanitary work, such as sewage disposal.

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There are also the State Immigration Department; State Department of Insurance; Labor and Industrial Commission; the Livestock Sanitary Board; the Nursery and Orchard Inspection Service; the State Nurses Examining Board; the Professional Engineers Licensing Board; the Department of the Public Examiner; the Department of Public Property; the Department of State Parks; the Railroad and Warehouse Commission with its technical staff; the State Securities Commission; the Surveyor General's Office; the State Tax Commission; the Game and Fish Department; the Reforestation Service; the Department of Weights and Measures; the Division of Banking; the State Sanitorium Advisory Commission; U. S. Department of Commerce Office; Board of Control; Feed Inspection; State Humane Society; State Inspection of Hotels; State Board of Investments; State Boiler Inspector's Office; the Board of Pardons; Department of Rural Credit; State Weed Inspector; and the State Law Library.

To add still further to the political atmosphere of a capital city, the Federal Government is here represented by the U. S. District Attorney and the Federal Court, the U. S. Marshal, Customs Office and Internal Revenue Officials, the Prohibition Administration Service, the U. S. Bureau of Animal Industry, the U. S. Agricultural Bureau of Markets, the Army and Navy Recruiting Offices, the U. S. Civil Service, U. S. Veterans Bureau and Veterans Hospital, the U. S. Weather Bureau, the Meteorological Station at the Airport, the U. S. Engineer's Office, the U. S. Bureau of Public Roads, the Secret Service, Public Health Service, Feed and Grain Inspection, Food, Meat, and Drug Inspection, Banks of the Federal Reserve and Land Bank System, and the U. S. Forest Experiment Station.

All the people in these departments have a governmental or political complex. They are interested in Government, its personnel and its method of doing business. Their engineering and architectural conceptions are based on governmental practices. They are a group who think along different lines and who have a different viewpoint from those engaged solely in trade and commerce.

A proper consideration of the housing of these departments leads to important and rare possibilities in the matter of location and design. All have distinct functions and require a definite type of office space. They should be located for the convenience of the citizens of the State who transact business in these departments and for the convenience of the officers themselves for inter-communication. In fact, a large number of the people going and coming by train, bus, automobile, and air have some business with the State or Federal Government, or its various governmental departments, and they travel to and from the capital city on Government business; their secondary interests are social, other interests and business being incidental.

The streets leading to the Capitol and Capitol grounds should be of special design. They should have a cross-section for two or three roadways, separated by park strips, two wide one-way pavements, wide sidewalks, ample boulevards—all carefully landscaped; or three roadways, one major traffic lane, with side service roadways to serve abutting buildings. This requires a way of considerable width which is necessary in a Capitol group. The buildings will be of a monumental character, the streets must be wide and spacious, the street appurtenances must be designed in a generous way, and the public utilities such as sewage, water, message, heat, light, and transportation require special engineering treatment. The natural development along such an esplanade would be buildings of public and semi-public character, such as club houses, churches, lodge temples, hotels, theaters, and small shops of a high-class character. Retail business, however, should be on parallel and cross streets, in order that it may carry on the same as in any city and develop its own characteristics and spirit of business, separate and distinct from the State Capitol development. These things are controlled by proper zoning.

To carry out the idea of a State Civic Center there must be whole-hearted co-operation between the State Department in charge of grounds and development for the State and the City Planning Department of the city. The thoroughfare plan of the city should be co-ordinated with the Capitol approaches and so designed as to create fine vistas. The zoning of traffic is also necessary so that heavy trucking and commercial vehicles may have streets of their own on which to travel, and not be required or permitted to move over the esplanades and semi-parkways designed for approaches to the State group.

The State Civic Center is a unique and interesting development, superimposed upon the commercial district of the city and so placed that it will not interfere with its commercial expansion. At the same time, it should be the show place within the business center or adjoining it.

Capital cities are peculiarly political regardless of their commercial or manufacturing interests. The social life is a political social life and the State's activities will more or less dominate the character of the city despite other conditions.

As pointed out by Mr. DeBoer, the people of a Republican Commonwealth are no longer interested in diamond-studded uniforms or decorations for their officials. They have transferred their admiration and allegiance to fine architecture as depicting the spirit, culture, vision, and economic importance of the State. They demand architecture of rare distinction in a pleasing setting. They are style conscious in architectural motifs and the design and placement of State edifices. The buildings must have beauty both inside and outside, and the grounds must give an air of spaciousness brought about by fine landscaping and the proper spacing of buildings, forming a major development of the city. The people of a capital city are beginning to recognize these things, and the people of a State are impressing it upon the people of the capital city that there must be full co-operation in the development of the Capital Civic Center, which will place both the city and the State before the world as built by a people of vision and engineering intelligence.

St. Paul is the capital city of Minnesota. It is a metropolitan city, and the fourth or fifth largest capital city in the United States. It has one of the finest Capitol plans of any city in the United States, except the Nation's capital—Washington. When Mr. Cass Gilbert was supervising the construction of the Capitol in 1903, he made tentative sketches of the possibilities of the approaches to Capitol Hill. In 1907 he was definitely employed by the City of St. Paul to prepare the State Civic Center Plan, so that the city itself in all its proposed street widenings, in the acquirement of down-town parks or public squares, and in the construction of public buildings, could co-ordinate them with the Capitol Plan. During the years intervening the State Historical Building was constructed and grounds have been acquired from time to time in accordance with this plan. Nothing has been done by the City of St. Paul which would interfere in any way with the complete development of the State Civic Center.

A State Office Building Commission authorized to expend $1 500 000 for housing State offices has referred back to the Legislature the question of location and number of buildings. This was brought about by State-wide protest against proceeding without first adopting a State Capital City Plan.

The question of the State's interest in a State Civic Center has been settled in Minnesota, and the future development of Minnesota's capital city will be made in accordance with a plan.

S. R. DeBoer,* Esq. (by letter).—The discussion by Mr. Knowles points out very correctly that, after all, a capital city is, in the first place, a city in which places to live, to make a livelihood, and to enjoy education and recreation must be provided. The presence of Government offices in a city is secondary to these general problems of municipalities at large, and in essence the question comes to this, that they form one of the many sources of livelihood that a city must have. The importance of the Government center decides whether or not its influence on the life of the city is an important one. There are in the United States a great number of small cities—State capitals—to which the State offices form one of the major single items. In the city's business life this must be fully acknowledged and to a certain degree the city should be built toward it. The average citizen of a Commonwealth may be ever so much opposed to taxation, but to have a capital city of which he must be ashamed is beyond his conception. In other words, it must perforce become a "dress-up" city that equals or surpasses others in the State in attractiveness and compares well with other capital cities.

Whether a capital city should not encourage industry and commerce must depend on the relative importance the Government offices hold in its business life. Nearly every capital city will find it to advantage to have some industry and commerce, and to such large cities as Boston, Mass., these overshadow the Government business to such an extent that the latter becomes of minor importance. Even in a small capital city, where the State offices are extremely important, industry can be safely encouraged if it is located in a carefully selected district. Any city is much better equipped by having many sources of livelihood rather than just one, but if the Government offices play the important rôle of being the main resource, the city authorities should fully recognize the fact and build to it.

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Denver is to a great extent a governmental city and the administrative offices play an important part in its business life. It is not only the Capital of the State of Colorado, but its citizens consider it more or less a Western Capital of the Nation. It has a very large number of Federal offices and a tremendous territory is administered to from this center. In Denver's business life the Government offices have become of unusual importance through the fact that climate and mountain scenery have made it a National recreation center as well, in a sense similar to Washington, D. C. Tourists form a good adjunct to the business life of Colorado's Capital City.

Mr. Knowles' reference to the industrial survey is timely. Too many cities have concentrated on getting industries at the expense of neglecting the building of a desirable city. The aim must be, in the first place, contented, healthy, and educated citizens. If for this purpose more industry and business are necessary, well and good, but to encourage the latter for the mere sake of greater population and higher real estate values is as absurd as it is frequent. In generations to come many cities which have overdone industrial efforts may have to demolish what has been built now, in order to create better living conditions. The writer agrees fully with Mr. Knowles that the city engineer was the first city planner. Many cities owe their conditions, good or bad, to the city engineer. He has considered every improvement in its value to the city as a whole, and he has done this work with a loyalty and earnestness that characterizes the profession. His main handicap has been the lack of a study treating the whole city as one entity, rather than many separate parts and in such a way that fundamental lines of development are laid down for a long period to come. The few engineers who did follow these lines must have realized the difficulty of getting elected councils to follow their imagination. The present age with its highly specialized work is more inclined to relieve the city engineer of the burden of city planning and leave this to specialists, and thus allow the engineer more time for his already complicated work.

In many respects Denver has problems that are the very opposite of those of other communities. Many cities on lake or ocean fronts are battling the problem of drainage. Denver is a mile above sea level and is the heart of the greatest semi-arid region in the country. Farm lands must be irrigated in order to yield heavy crops. The city is surrounded by large areas of these irrigated farms and they stabilize its growth and real estate values. Great care is exercised by the water authorities to see that in the growth of the city this farm water is not used for domestic supply. The domestic supply returns to the river bed several miles below the city limits; a use of the water from farms above would mean the shifting of irrigated land with a possible shifting in the growth of the city. Arrangements are being made to bring water through the Moffat Tunnel from the western slope of the Rocky Mountains for use in Denver. This water will not be needed for many years, however, and the additional storage now being planned will provide water from the eastern slope sufficient for probably 25 to 50 years' growth.

In Mr. Lyman's discussion the very interesting numbering system of Salt Lake City, Utah, is described. This system is undoubtedly of great practical value. Many cities have followed the plan of naming streets after their leading men. There is a more human factor in this practice than in mere numbering. It has often seemed to the writer that the names of streets should indicate to what they lead, such as Capitol Avenue, University Avenue, etc. This would not be practical for large numbers of streets, but would well be possible for the more important avenues.

The plan of Salt Lake City, with its blocks 660 ft. square and its streets 132 ft. wide, presents an unusual problem in city planning. The wide streets are of great value in present-day traffic. Just how far the interior of the large blocks will prove to be practical the future must tell.

In his discussion, Mr. Herrold confirms the thought that the writer tried to express, namely, that the people in a capital city must clearly understand its function; they must realize its position in the eyes of the citizens of the State, and they must strive to build to attain that position. A State Civic Center with broad parkways leading to it embodies considerable of this thought. Denver has undertaken this in its Civic Center, which is a continuation of the Capitol grounds. The two have been developed in such a way that a harmony exists, which makes a general State center out of them.

The suggestion embodied in Mr. Herrold's discussion of broad parkways leading to the capitol buildings and fronting them with buildings, such as lodges, churches, etc., is a very attractive one. The time will come, perhaps sooner than can now be predicted, that such semi-public buildings can be forced to a location of this kind by zoning provisions. Some zoning men frown at this statement, but the step to a definite regulation of this character, with many more of its kind, is much shorter than the original step to zoning.

The City of Cheyenne, Wyo., is working more or less in this direction. It is constructing all its lodge buildings and some of its churches on Capitol Avenue; it has plans for a State center. Some of it is a conscious effort and some perhaps subconscious, but the results are very promising.

In closing this discussion on capital cities it may be of value to recapitulate the governing ideas brought out:

First.—A capital city must recognize its position in the State and Nation and try to express the pride of the citizens of its political entity.

Second.—It must carry out this idea with, and not in spite of, its following the usual building program of any city.

Third.—It can encourage industries and commerce, but must be careful in their location on the plan of the city.

Fourth.—It must lead its State or Nation, not only in beauty of civic buildings, but also in beauty of residential and business sections.

Fifth.—It must lead where others can be contented to follow.

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Paper No. 1758

GOVERNMENT RECLAMATION POLICY IN BRITISH INDIA*

By E. S. Lindley,† M. Am. Soc. C. E.

With Discussion by E. R. Foy, Esq.

Synopsis

This paper describes the British Indian land-holding and revenue system based on ancient custom; how, in the Punjab, water is given, and charged for; what land is reclaimed, and how. Without attempting a comparison with America, attention is drawn to points that seem to be of value. Data are given which show the magnitude of irrigation and reclamation works which seems not to be realized in America.

Efforts of the British Government in the Punjab cover nearly all the reclamation work of India. In a pamphlet, "Federal Reclamation, What It Should Include", Elwood Mead, M. Am. Soc. C. E., devotes a section to "What Other Countries are Doing". In this, facts and even mention of India and Egypt are conspicuously absent. This is not an omission that is unimportant, for it is believed that the 27 500 000 acres annually irrigated in India are more than one-half the irrigation of the whole world. Of that in India, more than 40% is in the Punjab; the 5 000 000 acres of desert Crown Waste populated by canals constitute more than 20% of the area served by existing canals in the Province, and schemes under execution or projected will double that area of reclamation. Moreover, these data do not include privately owned desert land. In what follows, costs have been translated into terms of American money, for convenience.

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* Presented at the meeting of the Irrigation Division, San Diego, Calif., October 4, 1928.

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The Economic Problem and Its Solution

The reclamation policy of the Punjab Government has grown up from experience, starting from certain points of fiscal custom which must be described for proper understanding. Immemorial custom handed down from previous administrations prescribes that private "ownership" of land is subject to payment of the Government Land Revenue, or rent, as a first charge; in fact, in the last resort land may be sold to recover unpaid revenue. The owner may cultivate the land himself, or may let it to tenants. Such tenancies may be as firm as the ownership itself, subject only to payment of the agreed rent to the owner; or they may be ordinarily terminable at will.

Certain large desert tracts that were uncultivable because of their aridity were "Crown Waste", subject to no rights of ownership. Questions of "reclamation", called "colonization" in the Punjab, arise only where such Crown Waste is brought under irrigation.

The Punjab covers 65 000 000 acres, inclusive of mountains and areas with sufficient rainfall, of which, 21 753 000 acres are within the irrigation limits of Government canals already completed. Of this area, 4 726 000 acres were formerly Crown Waste; 11 160 000 acres was the average annual irrigated area (measured field by field) in 1922; and the tendency of growth is still upward apart from the opening of new canals. Canals now under construction in the Sutlej Valley Project provide for covering 8 878 000 acres, of which, 3 821 000 acres are Crown Waste; other canal projects which have been framed for successive execution will bring the area covered to a total of 35 327 000 acres, of which 9 674 000 acres in all are, or were, Crown Waste. The largest single block of Crown Waste was on the Lower Chenab Canal—2 300 000 acres out of a total of 3 391 000 acres covered; this canal irrigates more than 2 500 000 acres annually.

The customary revenue taken by pre-British Administrations was "in kind", by division on the threshing floor; the fraction taken varied, but was most commonly one-sixth of the gross produce, at least in theory. The British Administration soon based its demand on the net produce, and substituted cash rents; the fraction applied to this smaller amount was soon fixed in theory at one-half. The British claim may be described as one-half the economic rent in theory, but actually only from one-third to one-quarter is charged. The change to cash rents based on net produce obviously involved a system of assessment of agricultural profits. Such assessment, called a "settlement", is repeated at intervals, generally of twenty years.

Where the conditions affecting agriculture are relatively stable, the revenue to be levied is fixed and distributed over all the cultivable land; in bad years, its collection may be postponed, or even remitted. Where the conditions are less stable, the revenue is reduced to acreage rates to be levied season by season on the fields actually cropped; and in bad years the revenue may be reduced or remitted.

Proprietary lands to which irrigation is extended are generally under the latter ("fluctuating") form of assessment, because rainfall is uncertain. For land that can be given water, the "dry assessment" is then increased by a "water advantage rate", this being the revenue on the increase of rental value due to irrigation facilities. The total "wet" assessment for such land is now about $1.70 per acre in the year when cropped.

Water is also charged for at acreage rates per crop season, levied only when water is actually used; these average about $1.70, varying from $1.00 to $4.00 according to the crop grown. These rates are fixed partly on the value of the crop and partly on its water requirements. They are deliberately fixed very much below the commercial value of the water, which must approximate what it would cost a cultivator to water his land from a well with bullocks, say, $9.00 per acre per crop. On a small group of private inundation canals, instead of the water rate, the proprietors take "chauharmi", one-quarter of the divisible produce on the threshing floor.

There are no water rights enforceable at law, on the supplies delivered by Government canals. It is the policy of the Government to supply water to all whose land is so situated as to be able to take it, to such extent as water is available. This policy is followed impartially. Occasional proposals to penalize villages or individuals by withholding water, have invariably been disallowed, and frequent discontent is caused by refusals to reward loyalty or services by extra water. A project now under execution provides largely for irrigation in States under Indian Princes. Apart from the duty incumbent on any civilized administration, its interests are to exploit resources to the greatest benefit of the community as a whole. The degree of these interests can be judged from the fact that the net revenue derived from the construction of canals, a return of nearly 20% on the capital invested, constitutes more than 40% of the total revenue of the Punjab.*

When irrigation is extended to a tract, it is divided into zones according to the depth from the surface to ground-water. For each zone a percentage of the total area is fixed for which water shall be provided; it is lower for proprietary land (which can be cultivated to some extent without canals) than for Crown Waste to be reclaimed (which it has not been possible to populate at all without a canal). These percentages range from 25 for proprietary land with ground-water within 20 ft. of the surface, to 75 for Crown Waste with ground-water more than 40 ft. from the surface. The ratio of irrigation proposed in the summer and winter seasons is also fixed, as well as the summer and winter discharges at heads of farmers' ditches, per acre of proposed irrigation. Higher rates of discharges are given to some poor lands which need more water. These quantities, applied to the area to be served by each farmer's ditch, give the discharge to be delivered there; the outlets at the heads of farmers' ditches are designed to deliver these discharges. If the summer rate is higher than the winter rate, the outlet is designed for that, and in winter the time of delivery is reduced according to the supplies actually available. That the designed rates of supply are liberal is shown by the irrigation accomplished; this is 1.21 times the proposed irrigation for the entire system of canals in the Punjab for which supplies are secured by weirs (85% of the whole).

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* The Lower Chenab Canal, which brings in two-fifths of this revenue, pays more than 50%; but this is not profit calculated in a commercial way. Water rates alone minus working expenses are 10% for the whole Province and 22.5% for the Lower Chenab Canal. The remainder is accounted for partly by enhancement of land revenue for irrigation facilities, and partly by land sales; it is not clear whether the last is interest on total realizations to date, or instalments received during the year.

Before the recent introduction of "semi-modules" for outlets, it was impossible to determine the average discharge—the criterion in judging complaints of insufficient supply was then the area actually irrigated. If an outlet were found to be irrigating less ground than proposed, and this was not accounted for by waste or by intentional limitation to support the complaint, the size of the outlet would be increased (after approval by competent authority, and not merely by the local assistant engineer). Some engineers would even admit as the criterion not merely the proposed irrigation, but the ratio actually achieved in the neighborhood. With the introduction of "semi-modules" the criterion is becoming the discharge actually delivered, as it should be; this has the advantage of simplicity, and of no longer penalizing care and efficiency in irrigation.

When a new canal is opened, it is at first possible and necessary to give more than the designed supply to the land; this is gradually reduced as more land is developed. Under the conditions described, the originally designed area does not form the limit of development that is possible with a given supply of water. In some areas the ground-water level rises sufficiently to limit the supply that can be applied without harm, or even to water-log the land. In some cases actual irrigation has risen so far above what was proposed, as the result of improved distribution by the Government engineers (and, probably to a lesser degree, better irrigation by the farmers), that supplies can also be reduced. There is nothing to limit such reductions of present supplies, beyond the need to give fair and equitable treatment to present irrigators.

Experience with the Lower Chenab Canal

The Lower Chenab Canal Colony is a good example of Punjab reclamation. It had 2 250 000 acres of Crown Waste, and serious inaccessability. The small blocks that were colonized previously in other places had afforded some valuable experience, although reclamation problems can hardly be said to have arisen. Since then, other areas have been colonized, some of them considerable, but principally after the Lower Chenab Canal had become an assured success; these will be referred to in noting points of difference.

The Province was predominantly agricultural, about 90% of the population being occupied with farming, and the inhabited parts were sufficiently crowded to cause land hunger. Peasants, however, are conservative—where the margin between earnings and needs is narrow and precarious, an Oriental feels less the impulse to rely on his powers and try to better himself, than the fear of losing even what he has. "A bird in the hand is worth two in the bush". Canals had not then achieved the conspicuous success they now have, and a serious failure might have stranded the whole population nearly a week's march even from drinking water. However, the Administration did not need to go beyond its own borders to find settlers.

The total of settlers required was divided into quotas for each district, for which the Deputy Commissioners in charge of the Civil Administration were asked to nominate candidates. These quotas were presumably determined by relative degrees of crowding, and the quality of the peasantry. In some districts more difficulty was experienced in finding settlers than in others; in fact, it is often claimed that some districts made this an opportunity to export some of their undesirable characters; thus, the original quotas were not rigidly exacted.

In previous schemes settlers had also not been taken from the adjacent surroundings; in one case because of local opposition to the scheme; in the other because it was found that the attempt injured the proprietors by drawing off their tenants. In later schemes, it has gradually become a matter of selection from the crowd of eager applicants.

Settlers were sought for land grants of three orders of size: Peasants, to receive from one-half to two unit "squares" of 28 acres each; yeomen, to receive from three to five squares, which they would cultivate themselves with the assistance of hired labor; and capitalists, who were to receive larger grants to lease to tenants, and were intended to become the gentry and natural leaders of the new community. A small proportion of the area was sold by auction. In the first blocks, 67.5% of the area was granted to peasant colonists, 15% to yeomen, and 10% to capitalists. In later blocks, the first class was increased at the expense of the two latter, as these were found to be less satisfactory colonists.

The terms of course included payment of water rates, then about 80% of the present rates (but on a new canal water is always given free for the first few harvests); also land revenue, although this seems to have been only about $0.50 for the first years. In addition to this revenue, corresponding to the revenue in proprietary lands, there was a payment to the Government as the proprietor. Yeomen had to pay $2.00 per acre in two instalments, and capitalists from $3.30 to $6.60 on entrance; and both had the option to acquire proprietary rights after 5 years for $6.90 per acre. Peasants were given tenancies for 20 years, with the option of acquiring permanent tenancy rights within 5 years and without payment; they were charged an annual royalty, apparently only $0.125 at the start, in lieu of the rent they would have had to pay to a proprietor. About 20 years afterward, these peasants were given the option of acquiring proprietary rights for $4.20 per acre, payable in instalments.

In addition, peasant grantees were from the beginning required to reside on their grants. Very soon the same condition was laid also on yeomen grantees, and the building of a house was further explicitly enjoined. All were required to join in digging a village well, in keeping the village less unsanitary than is customary in the country, in protecting the common land (roads, farmers' ditches, grazing grounds, etc.) from encroachment, and in planting a few trees. To protect them from their own improvidence, rights of alienation were restricted; and to keep control of the type of settler the customary rules of inheritance were limited.

In effect, the settlers were given the bare ground to start on. Boundaries had been demarcated, a village site and roads were located, and canals and farmers' ditches had been made ready to bring water to a point in each holding. All else, from clearance of brush to the building of houses, was left to them.

In addition to the Deputy Commissioner appointed to ordinary civil charge of the new area, and the Canal Staff for technical duties and canal administration, a Colonization Officer of the Indian Civil Service was appointed for work in connection with selecting the settlers, allotting them their grants, seeing that they fulfilled their conditions, and all the work incidental to establishing the new population.

The writer has not ascertained the prices at which land was sold in the early colonization of this area. In 1896, when this colony was just beginning a rapid increase of irrigation, an average price of $17 per acre was realized in another small and fairly accessible colony; while in 1904 the average price of a further small auction in the same colony was $89 per acre. Now (1928), the average rent commanded by land on the Lower Chenab Colony is about $6.60 per acre, while good land favorably situated commands three times as much. About 1916 land in the Lower Bari Doab Colony was sold during a land boom, and on occasion more than $200 per acre was paid, for new unbroken land. The fee for which peasants were allowed to acquire proprietary rights had gradually been raised, and was then $33.30.

To a proportion of grants on the Lower Chenab Canal special conditions were attached; some grantees were required to maintain a small area planted in trees; some were required to maintain a suitable mare for breeding remount horses or transport mules, and to give the Army the first right of purchase. Many of the "Janglis", the former nomad grazing tribes who were the only inhabitants of the desert, were required to continue to maintain camels, and supply transport for Army use when needed. In the Lower Jhelum Colony the bulk of the grants were made on horse-breeding conditions. An effect of this has been that the settlers are, on the average, a less satisfactory type of farmer.

In the earlier schemes, a proportion of the grants was earmarked for ex-soldiers and the races with traditions of military service. The colonization of the Lower Bari Doab Canal was still proceeding when the main recruiting effort for the European War was made, and to assist this the area earmarked for military grants was considerably increased. Including the scheme now under execution, a total of 387 000 acres has thus been specially set aside or given in military grants. This area does not include the grants mentioned previously, or special good service rewards of land given free of revenue demands.

In the earlier schemes only small areas were auctioned. As colonization of the Lower Bari Doab Canal was still proceeding when the period of difficult war, and early post-war, finance was entered, land sales were increased to meet the difficulty. In the scheme now under execution, land sales are providing an appreciable proportion of the capital needed for construction, thus decreasing the amount that has to be raised in loans.

The experience and now assured success of the older colonies make a modern colonization simple when compared with earlier efforts. Some of the more obvious difficulties can easily be imagined. Before the colonists on the Lower Chenab Canal had settled down at all, a severe epidemic of cholera swept over them, and the Lower Jhelum colonization was interrupted by plague. If allowance is made for picturesque description, some of Kipling's stories convey an idea of such situations, with a handful of Englishmen and their womenfolk heading a staff of Indians to manage a scared crowd. The Janglis did their best to wreck the scheme; they claimed ownership of the tracts they had roamed and grazed over, and the earlier administrators too lightly ignored the question of what to do with them, although the nomads of other deserts naturally would not admit them. One of their standing occupations was cattle theft, and by this and in many other ways they persecuted the settlers. This drove many away, and others frankly declared their intention of leaving as soon as free water ceased. However, the scales were turned when some years of good rainfall up to 1894 were succeeded by lean years from which the canal protected the tract; relatives and labor flocked in, and the settlers began to make good profits. Soon afterward a railway into the colony was opened and this improved the marketing of produce.

An internal question in this work of colonization and canal administration has been the fairness of the terms charged. The first colonization was so experimental that it was necessary to offer extremely liberal terms at the start, and to think more of the point of view of the settler than that of the taxpayer. As success was achieved, it became possible to take a wider view, and terms have gradually been raised. The question is debated from two points of view: The settlers' party asserts that the high rate of profit from these works means that they are being called upon to bear an undue burden; the opposite point of view is that these profits are really a measure of the extraordinarily favorable conditions for canal development, and that the charges are only a fraction of the true value of what is given. The settlers are drawn from the agricultural population of the whole Province, which is the bulk of the whole population; and, much of the Province being under canal irrigation, the bulk of the agriculturists are equally interested in low water rates. Opposition should come from those who are not in a position to enjoy cheap water and colony land—those whose taxation would be lightened if the charges were raised. Actually, this opposition is hardly vocal, and the Government has at the same time to voice the opposition and to judge the issue fairly. As a retired official of that Government, the writer simply presents these statistics without personal comment.

Lessons Learned from Indian Reclamation

Without attempting comparisons with American conditions, it seems worth while to draw attention to some points that appear of value.

India has the obvious advantage, in its agricultural population, of a reservoir of settlers of the right stamp. Success has been greatly facilitated by the liberal terms given them; but these were made possible only by favorable natural conditions—ample supplies of water which could be brought cheaply to large blocks of flat fertile country.

By comparison with provinces in which canal engineers were restricted to the technical maintenance of channels, it has been found advantageous to make the engineers responsible also for revenue work, as that brings them into intimate touch with the results of their technical labors.

Instead of the canal management being in the hands of the farmers, who need consider only their own interests, it is in the hands of the Administration which is impartial. The branch of the Administration immediately concerned is often accused of bias, of considering too much the showing of profits to mark its efficiency; but it has not the sole voice in the Administration, and the data quoted make it possible to judge whether there is bias shown. Impartial management is an advantage toward securing efficiency.

Given an impartial administration, it is an advantage to have water rights in such a form that fair development is not checked. The system of water rights described is advantageous in permitting the exploiting of supplies greatly in excess of the minima.

It is disadvantageous that irrigation receipts go straight into the general revenue, and that funds for improvements depend on the general budget situation. This disadvantage in the case of Indian State Railways has recently been admitted and removed.

DISCUSSION

E. R. Foy,* Esq. (by letter).—The author gives a clear and interesting exposition of "colonization" as carried out by Government agency in British India. The problem of installing settlers on new lands in that country is shown to be a State function. The chief interest of this paper, therefore, lies in comparing methods used in India with those in the United States.†

In India, the control of all water rights is vested in the Supreme Government. Due regard, however, is being paid to the rights of the Native States which are governed by Indian Princes. If there is any clash of interests the matter is adjusted by arbitration. The projects for supplying irrigation water, either to old proprietary lands, or to new areas that are being colonized, are financed by the Government. An adequate income is obtained by means of water rates levied on each crop-acre irrigated, and by the sale of new lands to settlers. These projects are generally remunerative, but there are a few areas in which canals are "non-productive" and in such places irrigation projects are undertaken as protective measures to guard against periodic famines. In years of good rainfall, irrigation water is not needed, but there are years in which failure of the seasonal monsoon rains causes failure of all husbandry operations, thereby inducing heavy demands for canal water and thus increasing the income of these otherwise unfavorable projects. It must be realized, however, that no colonization programs are undertaken in areas where cultivation with natural rainfall is possible. Protective works are only provided for occupied or proprietary villages subject to periodic famines.

In America, it appears that at first there was no settled policy, and the earlier irrigation projects were the result of private enterprise. Some of these were for the mutual benefit of the local landholders and settlers, and others were purely speculative ventures. In some cases speculators disappeared as soon as they "had made their pile," leaving those unfortunates who had invested their money to carry on as best they could. Now, however, the Federal Government seems to be assuming complete control; but even on State irrigation projects the procedure does not appear uniform, and varying conditions prevail. Some settlers are required to pay full charges and costs, while others have special privileges as subsidies, remission of interest dues, etc. Assumption of complete control by the State, however, is a forward step and no doubt a uniform policy will be developed in due time.

When the writer was, in 1897, first posted to the Lower Chenab Canal Colony in the Punjab, "colonization" methods were just emerging from the experimental stage. Water was then plentiful, while land was available on absurdly liberal terms; but the difficulty lay in inducing settlers to break ground under adverse conditions in new surroundings, even when it held promises of wealth as an alternative to starvation in their old homes. After some years of slow progress, the Administration was just beginning to gain the confidence of the settlers. Since that time two other projects involving several millions of acres have also been completed and are now fully matured. There is now (1929) a fourth irrigation project being brought to completion, and there is a wild scramble for land at any cost. In the Sutlej Valley Project, 9 000 000 acres of new and old lands are being developed to expand irrigation in British India and in the two Indian States of Bahawalpore and Bikaneer. Water was turned into one of the series of these canals on this project in 1926. Ultimately, this part of the project alone will develop 1 000 000 acres of new land in British India, and more than that in the State of Bahawalpore. It will also insure an adequate supply to a large area of old village lands in both territories. The settlers of to-day find it hard to understand the hesitancy of their predecessors thirty-five years ago in accepting the first farm lands offered on these projects.

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* Cons. Engr., Ilchester, England.

† "A National Reclamation Policy: Explanatory Statement, Regarding Report of the Committee of the Irrigation Division," by J. B. Lippincott, M. Am. Soc. C. E., and "A National Reclamation Policy: Economic Aspects of Federal Reclamation," by Elwood Mead, M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, pp. 1193 and 1207.

Mr. Lindley refers to the political side of the question concerning high *versus* low rates. Fortunately, irrigation is not what is known as a "transferred" subject under India's "reform" administration. The ultimate decision on matters of policy rests with the Government and not with the Legislative Council; therefore, the danger of interference by political parties is not yet present.

Apparently, political influence is a factor in America. An eminent member of the U. S. Reclamation Service once made a short tour through the Punjab. In the course of conversation with the Governor of the Province the visitor said, "Keep irrigation out of politics. Do not give the farmers a vote." Coming as this did from a prominent citizen of democratic America, the Governor was rather surprised. The visitor then went on to explain that some rather good and financially sound irrigation projects in America had been either wrecked or quite severely hampered by candidates for election to Congress, who secured the settler's vote by promising either cancellation of the unpaid balance or postponement of payment if elected. This condition is verified by J. B. Lippincott, M. Am. Soc. C. E.,* although he further explains that the State is beginning to adopt a more rigid attitude toward those who fail to pay.

An important detail not mentioned by Mr. Lindley is the absolute necessity of providing an ample communication system on irrigation projects before the arrival of the first settler. Of primary importance is the construction of a railroad. Next, all the main arterial roads and highways; and, third, the "feeder" roads to connect the main highways, the railway stations, and the near-by markets. These obvious conveniences were neglected forty years ago when the Lower Chenab Colony was first being developed, and no doubt that factor contributed greatly to the lack of enthusiasm for the new lands. In the latest of the developments (the Sutlej Valley Project) an existing railway was re-aligned so as to pass through the middle of the new colony, while a complete and carefully designed system of roads and main highways is being provided, and special marketing towns have been located at suitable points along the railroad. In these towns the competition to secure building sites is particularly active and splendid prices are being realized. The extra cost of these conveniences is being recouped by the higher prices that the settlers willingly pay for the service, because they are no longer harassed by the difficulties of produce transportation, and isolation from their old homes.

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* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1193.

The writer is fully in accord with the lessons with which Mr. Lindley concludes his paper.

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Paper No. 1759

REFLECTIONS ON THE STATUS OF THE ENGINEER

ADDRESS AT THE ANNUAL CONVENTION AT CLEVELAND, OHIO, JULY 9, 1930

By J. F. Coleman,* President, Am. Soc. C. E.

For years there was a constant cry to the effect that the engineer did not receive the recognition which he deserved. Lately, the same cry is heard although less frequently. In the past there was much to justify such complaint, and even now there is some excuse for it.

It seems probable, however, that the engineer himself is in great degree responsible for such a state of affairs in that he has been until recent times almost inarticulate in the councils of men; and, at this date, while by no means silent, is heard far less frequently than he should be. It is astonishing how few laymen know what an engineer is and how he accomplishes the tasks allotted to him. For that situation, the engineer is more to blame than the layman; and it would be distinctly to his advantage to have others know more of him and his work than they now do. It may therefore profit the engineer to analyze the matter and to take steps to overcome this disability.

In the Public Light

Leaders of the profession have preached for years on the desirability of its members taking part in civic and other public affairs, and as time has passed, these preachings have borne some fruit. The engineer is to-day more prominently in the public eye than he has ever been before, and without doubt this improved position has been brought about at least in part by those engineers who have given of their time and their talents to public affairs.

It has been stated that in the 1929 edition of "Who's Who in America", there are 2 858 engineers and architects, among which number there are or have been 10 Governors, 13 Members of Congress, 2 Members of the Cabinet, and the President of the United States. There are also engineer presidents and a number of other engineer executives in large and important corporations. The profession is represented in chambers of commerce and other similar organizations and, on the whole, enjoys a better position than it formerly did.

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Advertising the Profession

Notwithstanding these facts, there appears to be an almost abysmal ignorance on the part of the layman as to what an engineer is and does. He knows that the engineer builds bridges, but he has no conception as to the designing of such structures, which must precede the construction. He knows that the engineer builds subway systems, railways, manufacturing plants, and water-works, but his information does not extend to any conception of what the engineer must know and do to plan such works in an orderly and intelligent manner before the actual construction may begin.

From his contacts with the doctor in the sick room or with the lawyer in the court room, he has some idea as to how the members of those honorable professions function. He is practically never present, however, at any stage of the work of the engineer, except that in which the construction is in progress and manned almost wholly by laborers skilled and unskilled, and, therefore, he has no visual evidence of the guiding hand and brain of the engineer who planned the structure and who directs the details of its execution.

It is a fact that in his daily life the layman is in more frequent contact with the products of the brain and skill of the engineer than with those of any other calling, but he takes these things as much for granted as he does the sunshine and the rain. The water in which he takes his bath; the electricity and gas which furnish light and fuel; the transportation agencies which bring his food to his door; the street railway, the automobile, or the railroad, which conveys him to his office; the office building itself, with its equipment of elevators, heating system, etc. All these are the products of engineering achievement as are countless other items of daily use, such as the telephone, the telegraph, the radio, and the airplane; but so far as conscious thought goes, to the average layman they "just happened", and he uses them without thought as to how they happened.

Plagiarizing of a Good Name

The World War was often spoken of and written about as an "Engineers' War", although it would be hard to find in any such spoken or written statement any information as to what made it such. As engineers, we have knowledge of those factors which caused it to be so designated. The various agencies whereby large bodies of men and large volumes of supplies were transported and handled; the various means of rapid communication, such as the telephone, the telegraph, and the radio; the submarine and the airplane; guns both large and small; and explosives of all classes and for all purposes. All these and countless other items important in war represent engineering achievement.

By reason of the prominence given to the name, "Engineer", during the World War, this designation attracted the favorable attention of people of all classes, some of whom showed a desire to capitalize on the public favor by calling themselves engineers or by attaching the word, engineer, to the name of their trade or calling. We thus had Forgery Prevention Engineers, Milk Engineers, Feed Plant Engineers, Socio-Religious Engineers, Display Engineers, etc., *ad infinitum.*

While the profession may have suffered to some extent from such practice on the part of these unauthorized persons, it is certain that it was extensively advertised, and it seems fair to presume that it gained as much as it lost.

Handicap of the Modest Temperament

Without doubt, each profession suffers from some disadvantage by comparison with others. Among those from which the Engineering Profession suffers and must continue to suffer is the fact that so large a percentage of its members are salaried employees. Whatever his capacity, the salaried employee must attain a position of some importance in the organization to which he is attached before he can expect to receive much consideration outside that organization. In his contacts with other men, he is handicapped by being a subordinate. He cannot be the master of his own time, nor can he be as free and independent in the expression of his views as are those who both work and speak for themselves. Under such circumstances, he is far less likely to seek or to accept opportunity for public expression of his views than is the man of any calling who is not on anybody's payroll.

In the performance of his professional work, the engineer is far more accustomed to the quietude of his office, his drafting room, or his home, where he analyzes data, studies reports, and the like, than he is to making public talks or speeches. His whole training and experience makes him somewhat hesitant in presenting his ideas until he feels that he has fully matured them and is in possession of all available facts.

These considerations being understood, it is easy to see why the engineer has been more or less inarticulate, and it is surprising that he has progressed as far as he has into the public consciousness. No man or set of men will ever get very far or make much progress if satisfied with the *status quo.* A healthy state of dissatisfaction is necessary to progress in any branch of human endeavor. The somewhat remarkable progress of the Art of Engineering bears testimony to the fact that the engineer has that healthy state of dissatisfaction.

While no such progress has been made in the status of the engineer as in the state of the art, it does not indicate a satisfied state of mind with respect to the status. It is rather an indication that the engineer has a more lively interest in his work than in himself.

Duty of Improving the Status

In an attempt to bring about some improvement in the status of the engineer, the American Society of Civil Engineers has within the past few months adopted a program for the broadening of its activities so as to reach into fields not hitherto covered by it. Committees have been appointed and have begun to attack the problems assigned to them, so that work is now in progress in a number of ways looking toward the desired end. The task is not an easy one, nor is it so definite in its nature as are many of the more technical ones which the engineer must approach. Also, it is a task which cannot wholly be accomplished by any number of committees. The task of the committees is rather one of determining upon the best means to be adopted to achieve results.

Each individual member of the profession has a duty to perform to attain the hoped-for ends. Each man may be and should be a "focus of infection" to his friends and neighbors and to others with whom he comes in contact. He should avail himself of reasonable opportunities to convey to the minds of others those things which the layman should know of the work of the engineer. To be most effective, there should be avoidance of anything like boosting or advertising of individuals in the interesting bits of information presented.

There are many most interesting facts which could be conveyed to laymen in ordinary conversation, or in brief talks, which would tend to create in the minds of the hearers a better understanding of the mental equipment necessary to the engineer, and opportunities for so doing are plentiful without the necessity for creating them.

Personal and Professional Rewards

Those who avail themselves of such opportunities are almost certain to reap individual reward even though they do not seem to seek it. Every man who conveys an interesting bit of knowledge to a fellow man attracts some measure of attention and respect from that fellow man, and such respect and recognition by one's fellows in sufficient number will assuredly result to advantage in a material way.

The engineer is as anxious for opportunity to improve his earning capacity as any other man. He is as ready appreciatively to accept added income and better surroundings as the next one. It has been charged that he is not as ready and willing to assume civic duties which do not carry financial returns. It is more likely, however, that he is not tendered such duties so frequently as are the representatives of other fields of endeavor, and this for reasons which have hereinbefore been noted. Such assignments are not infrequently the means whereby a man may disclose his capacities, and they sometimes result in his advancement.

The engineer is apparently often overlooked in the community in which he resides, and he has always seemed less ready to push himself forward and to express his thoughts than are men in other avenues of life. It is only when a man has achieved some degree of prominence that his words are likely to be heard with respect and attention; and the engineer being in the mass modest, or one might say super-modest, appears to hesitate about taking issue on general subjects with other men who either have or assume positions of greater prominence.

The Society's Opportunity

Possibly local sections of the National Engineering Societies and local engineering societies may aid in bettering things. They may be able to foster a proper public spirit among their members, and no doubt there are opportunities for them to put forward men for civic and other public work who will reflect credit upon themselves and upon the profession. The engineering societies and individual engineers should hold themselves in readiness to grasp opportunities of this nature as may come to them.

The American Society of Civil Engineers has now definitely set its feet in this direction and without doubt will continue in that course. With a reasonable amount of well-directed aid on the part of the individual members, it seems safe to predict good results. It is to be hoped, however, that as a profession and as a Society, we shall never become wholly "satisfied", as in that would lie inertia, atrophy, and, ultimately, dissolution.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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Paper No. 1760

CONSUMPTIVE USE OF WATER IN IRRIGATION

PROGRESS REPORT OF THE DUTY OF WATER COMMITTEE OF THE IRRIGATION DIVISION*

WITH DISCUSSION BY MESSRS. HAROLD CONKLING, CHARLES H. LEE, AND M. R. LEWIS.

SYNOPSIS

This report proposes detailed definitions of the consumptive use of water in irrigation. It also calls attention to differences between the basic meaning of consumptive use and its application to the farm, the project, and the valley. Common difficulties in making a dependable determination of consumptive use are enumerated and briefly discussed. Experimental observations of consumptive use of water in typical irrigated valleys of the West are reported.

INTRODUCTORY

The rapid growth of American irrigation during the first quarter of the Twentieth Century has developed a keen public interest in a study of the disposal of irrigation water. The pioneers in irrigation had little opportunity fully to ascertain what became of the water which they applied to their lands. That they lost some water by surface run-off was obvious; that some water was absorbed by the crops they grew was likewise apparent; but that large quantities of water percolated deeply into the dry soil was to them merely speculation, if, indeed, such losses were suspected at all. However, the gradual rise of water-tables, with resulting enlargement of natural springs and the development of new springs, and the seepage return to flood-water channels, to small creeks, and, finally, to rivers, gave increasing evidence concerning the magnitude of losses of water through deep percolation. Moreover, it was found through experience that much less water need be applied to the farms to produce profitable crops than was formerly believed necessary, and the areas of land successfully irrigated by the water from a given stream were greatly increased without any apparent increase in the water supply. Obviously, such increase in area of irrigated land could not continue without limit, some water being actually consumed by the growing crops.

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* Presented at the meeting of the Irrigation Division, Denver, Colo., July 14, 1927. This report represents a part of the results of the work undertaken by the Duty of Water Committee of the Irrigation Division. A progress report on this subject was presented at the meeting of the Irrigation Division at Salt Lake City, Utah, July 9, 1925, by O. W. Israelsen, Assoc. M. Am. Soc. C. E. The present report has been prepared by Prof. Israelsen and includes, in addition to the material previously presented, further experimental data on consumptive use in different areas and analysis of the results.

Results of the work of the Committee on the "Determination of Duty of Water in Water-Right Adjudications" were published as Paper No. 1620, in *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 1074.

The membership of the Committee on Duty of Water is as follows: S. T. Harding, *Chairman*, Harry Barnes, Lynn Crandall, Augustus Griffin, Charles R. Hedke, O. W. Israelsen, R. I. Meeker, H. M. Murdock, R. L. Parshall, W. L. Powers, G. E. P. Smith, and O. L. Waller.

Definitions and Analysis

Gross duty of water, head-gate diversions, and net duty are familiar terms concerning water uses, but "consumptive use" is a term of recent origin, and should not be confused with the net duty on land and other terms used in discussions of duty of water.

Some engineers have used the term, consumptive use, as restricted to "valley consumption", or net depletion of river flow, whereas others have used it to include water used from all sources, or total water consumption, irrespective of whether it was from river water, rainfall, carry-over soil moisture, or absorption from ground-water. It is believed that the use of the definitions given in this report will avoid confusion in the study of consumptive use of water, and comparisons of engineering data will thus be placed more nearly on a common basis.

The term, consumptive use of water, was used in Colorado nearly two decades ago by John E. Field, M. Am. Soc. C. E., then State Engineer. It has since been used in Court testimony concerning water supply studies in the Laramie River inter-state water litigation between Wyoming and Colorado; in the North Platte River Co-Operative Report between the U. S. Reclamation Service and the State of Wyoming; in the Rio Grande Basin water supply studies of the U. S. Reclamation Service; and in the Colorado River Compact deliberations in the several Western States. It has heretofore been defined by R. I. Meeker, M. Am. Soc. C. E., of this Committee, as:

> "The permanent water loss incident to irrigation of large tracts of land. Generally speaking, consumptive use is the difference between the inflow and the outflow below any specific area under consideration, or the difference between the amount of water diverted and the amount returning to a stream. With respect to an entire river system, it is the net amount after deducting the outflow from the inflow."

To add convenience and brevity to the following definitions and analysis of consumptive use proposed by the Committee, mathematical symbols are employed and mathematical methods are followed, despite the fact that the equations are of a comparatively simple form.

Symbols Used.—The meaning of each of the more important symbols is fully given subsequently. For convenience of reference, the meaning of these symbols is briefly as follows:

Let U = consumptive use.
U_f = farm consumptive use.
U_p = project consumptive use.
U_v = valley consumptive use.
Q_h = quantity of available heat, in day-degrees, during crop year.
e = evaporation from water surfaces.
m = mean seasonal soil moisture content.
s = the soil with all its influencing factors, notably potash, phosphorus, lime, humus, and nitrifying power.
c = the kind of crop.
y = the yield of crop.
D_f = deep percolation losses from the farm.
H_f = the total quantity of water supplied the farm.
R_f = surface run-off from the farm.
D_p, H_p, and R_p = project deep percolation, project water supply, and project run-off, respectively.
E_p = project evaporation losses.
E_v = valley evaporation losses.
D_v, H_v, and R_v = valley deep percolation, valley water supply, and valley outflow, respectively.

Consumptive Use in a Basic Sense.—The consumptive use, U, is here defined as the quantity of water, in acre-feet per cropped acre per year, absorbed by the crop and transpired or used directly in the building of plant tissue, together with that evaporated from the crop-producing land.*

The direct source of "consumed" water is the water in the soil in any form that crops can absorb it. The indirect sources of "consumed water" are those parts of the precipitation and of irrigation water which are stored in the soil in such depths and for such time as to permit direct absorption by crop roots, together with such water as may be obtained by the crop roots from the ground-water after such water is raised to the plant root zone by capillary action. The consumptive use as herein defined is, therefore, equal to the total evapo-transpiration plus the water used in the building of plant tissue.

It is natural to inquire concerning the factors which determine the magnitude of U. A statement of the major influencing factors is made in the following equation:

$$U = \int (Q_h, e, m, s, c, y)\dagger \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

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* The consumptive use, as here defined, is not rigorously a correct application of the term, "consumptive," since the water which passes out of the crop leaves is not truly consumed. It is merely converted by the plant and the energy of the sun from the liquid to the gaseous phase. However, for the purposes of this report, water vapor in the atmosphere is considered beyond direct recovery by practical means, and hence the conversion of liquid water into water vapor is considered herein as consumptive use.

† Plant diseases and insect pests may reduce the crop yield, y, without proportionately reducing the consumptive use, U. Likewise, inefficient farm practices may change the relation between U and y.

It is apparent that most of the factors which influence the magnitude of U are themselves variable. The number of variables may be reduced somewhat under specified conditions; for example, if the soil of a farm is liberally supplied with organic matter and all the essential plant-food elements, so that the yield of a particular crop will be limited by the quantity of heat available and not by lack of moisture or plant food, it is possible to eliminate the last four factors in Equation (1) and write:

$$U = \int (Q_h, e) \qquad \text{(1}a\text{)}$$

In the light of the foregoing statements, it is unreasonable to expect that a precise experimental determination of U will give a definite magnitude that can always be considered rigorously correct. Engineers must use dependable approximations of the magnitude of U.

Knowledge concerning the magnitude of U is dependent on further analysis. Does U indicate consumptive use per acre on a particular farm, an irrigation project, or in an irrigated valley? Further definitions are clearly necessary.

Let U_f = "farm consumptive use";
U_p = "project consumptive use"; and
U_v = "valley consumptive use".

Definitions of each of these three modifications of consumptive use are considered desirable and are, therefore, proposed herewith.

It seems reasonable to assume that the public may expect the individual irrigator to account for all the water which goes on to his farm. With respect to a particular farm, the water applied is disposed of by consumptive use, plus surface run-off, plus losses from the farm by deep percolation.

Farm Consumptive Use.—Let D_f = deep percolation farm loss, in acre-feet per cropped acre per crop year. This loss usually cannot be measured with precision by direct means. Then, by definition, as used in this report,

$$U_f = U + D_f \qquad \text{(2)}$$

It is important to distinguish between U_f and a related quantity, H_f, which is defined as follows:

Let H_f = the sum of: (*a*) The quantity of water delivered at the farm plus (*b*) the water taken from moisture stored in the capillary form before planting, or during the non-growing season plus (*c*) the crop-year rainfall plus (*d*) the amount absorbed from the ground-water—all in terms of acre-feet per cropped acre per crop year.

Let R_f = surface run-off from a farm, in acre-feet per cropped acre per crop year. Then, by equating the water going on to the farm plus rainfall and draft on soil moisture plus the quantity absorbed from the ground-water, (H_f), to farm consumptive use, (U_f) plus farm run-off, (R_f):

$$U_f = H_f - R_f \qquad \text{(3)}$$

Equation (3) gives an indirect means of arriving at the farm consumptive use.

Project Consumptive Use.—In order to define U_p and indicate an indirect means of measuring it, the following quantities are introduced:

Let D_p = the non-measurable deep-soil percolation and capillary losses, in acre-feet per project acre per crop year, from any project.

Let H_p = the sum of: (*a*) The area of measurable surface water which flows into the project plus (*b*) the crop seasonal draft on soil moisture plus (*c*) the depletion of ground-water plus (*d*) the crop-season rainfall, also in acre-feet per project acre per crop year. Items (*b*) and (*c*) may be positive or negative.

Let R_p = the sum of: (*a*) The surface run-off losses, in acre-feet per project acre per crop year plus (*b*) such deep-soil percolation losses as contribute directly to a measurable rise in the height of the water-table plus (*c*) the deep-soil percolation losses as may re-appear in natural or artificial channels at the boundaries of a project.

Let E_p = the evaporation losses, in acre-feet per project acre during the crop year, from the non-cropped area within the boundaries of the project, such as roadways, barnyards, towns, etc., together with the transpiration from natural non-crop vegetation within the project boundaries.

Then, by definition,

$$U_p = U + D_p + E_p \tag*{(4)*}$$

By equating inflow to outflow plus change in moisture content:

$$H_p = (U + D_p + E_p) + R_p = U_p + R_p$$

and, therefore,

$$U_p = H_p - R_p \tag{5}$$

Equation (5) gives an indirect means of determining the project consumptive use. It is important to note from Equation (4) that the project consumptive use includes two factors which are difficult to determine directly, namely, D_p and E_p. With poorly prepared land, porous soil, small irrigation streams, and great distances between the head ditches, D_p is likely to become excessive, possibly greater than U. It is not probably extravagant to state that, in general, project managers should endeavor to reduce $(D_p + E_p)$ to a minimum and thus decrease U_p until it closely approaches U. On some high land projects, D_p may be recovered and used on lower neighboring land. On such projects it is less objectionable to have D_p large although excessive deep percolation losses on high land usually decreases the productivity of lower areas.

Valley Consumptive Use.—In order to define U_v and indicate an indirect means of measuring it, some further quantities are needed.

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[*] Provided a relatively small area of the land within the boundaries of an irrigation project or valley is irrigated, the common practice of determining project or valley consumptive use by dividing the difference between the inflow above and outflow below the tract in question by the area of the tract is misleading even if the corrections for contribution of rainfall, draft on soil moisture, and absorption from ground-water are properly made. To illustrate:

Let W = the total water diminution on the tract, in acre-feet;
A_c = the cropped area, in acres; and,
A_p = the area within the project boundaries, in acres.

From Equation (4),

$$W = U\,A_c + (D_p + E_p)\,A_p \tag{a}$$

Dividing Equation (*a*) by A_p,

$$\frac{W}{A_p} = \frac{A_c}{A_p}\,U + (D_p + E_p) \tag{b}$$

If the ratio, $\frac{A_c}{A_p}$, is small, approaching, say, one-half, then $\frac{W}{A_p}$ is not equal to U_p, as here defined. Equation (*b*) shows the importance of measuring A_c with a fair degree of precision.

Let D_v = the non-recoverable deep-soil percolation losses during the crop year recorded, in acre-feet per acre, based on the total area of irrigated land in the valley, that is, on the sum of the gross areas of projects within the valley.

Let E_v = the evaporation and transpiration losses from the non-cropped area of the valley per crop year, in acre-feet, divided by the gross area of irrigated land in the valley.

By definition as here used:

$$U_v = U + D_v + E_v \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

However, as in the case of a project, it is very difficult to measure D_v and E_v by direct means. This is particularly true of D_v, and, hence, other more easily measured quantities are introduced.

Let H_v = the sum of the following: (*a*) The acre-feet of water entering the valley in measurable channels during the crop year; (*b*) the crop-year draft on soil moisture; (*c*) the crop-year rainfall; and (*d*) the water derived from the ground-water divided by the irrigated valley area, as defined under D_v and E_v.

Let R_v = the measurable outflow, in acre-feet, divided by the irrigated valley area, as defined for D_v and E_v.

By equating inflow plus rainfall plus draft on soil moisture and ground-water (H_v) to valley consumptive use (U_v) plus outflow (R_v):

$$H_v = (U + D_v + E_v) + R_v - R_v + U_v$$

therefore,

$$U_v = H_v - R_v \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

It is very important that consumptive use as here defined, either in its basic sense or as measured on a farm, a project, or in a valley, should not be considered as representing the quantity of water which the farmer, the project manager, or the valley community should be permitted to divert for irrigation purposes. In localities of very low crop-year rainfall, deep loam soils, favorable topography, reservoir storage for water when not needed, and other favorable conditions, it is possible that the quantities, R_f, R_p, and R_v, will be reduced to a minimum, thus making H_f, H_p, and H_v approach U_f, U_p, and U_v, respectively, as Equations (3), (5), and (7) indicate would be the case. In general, however, such favorable conditions do not exist, and, consequently, there is no simple relation between the quantity of water necessary properly to irrigate a farm, a project, or a valley, and the respective consumptive uses.

COMMON DIFFICULTIES

Some of the more common difficulties in measuring consumptive uses under the cases considered, together with notes concerning their variability, are briefly given, after which some actual measurements are reported. The factors which cause variability in U at a particular place have been given in Equation (1). It is obvious that all the factors in the right-hand member of Equation (1) vary greatly from place to place. Based on elaborate experiments in Nebraska, Kiesselbach (10)* concluded that "there is no such thing as a definite water requirement which is constant for any kind of crop";

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* Reference is made by number to literature cited at the end of the report.

and, similarly, there is no definite consumptive use. In Equation (2), D_f depends on:

(*a*) The quantity of irrigation water applied in each irrigation;
(*b*) The uniformity of distribution;
(*c*) The frequency of irrigation;
(*d*) The length of land irrigated in a single run and the size of stream used;
(*e*) The texture of the soil and subsoil;
(*f*) The depth to the water-table;
(*g*) The dryness of soil between the water-table and the zone of root action, provided the water-table is very deep;
(*h*) The kind of crop and depth of root zone; and,
(*i*) The moisture conductivity of the soil and other less important factors.

In many localities, D_f was large during the early years of irrigation, but it is now relatively small because: (1) The dry desert soil to great depths has been fully moistened through the years of irrigation; (2) the land is better prepared for applying water; and (3) general improvement in irrigation methods has been made. The quantity, D_p, of Equation (4) is likely to be larger than D_f of Equation (2) since it includes deep percolation from canals, ditches, streets, yards, ponds, borrow-pits, and sloughs. It may be high on one project and negligible on a neighboring lower project.

To measure the evaporation and transpiration losses from non-cropped areas by direct means the engineer can measure the areas for which such losses occur and make an approximation. Relative to the $(U + D)$-quantities, such quantities are usually very small.

It seems advantageous, however, to endeavor to approximate the consumptive use on the farm, project, and valley areas by indirect means as indicated in Equations (3), (5), and (7). More detailed consideration of the difficulties of measurement and sources of error in the elements of moisture supply and run-off are, therefore, desirable.

Provided the farm, project, or valley considered has a deep uniform soil and deep water-table, the elements of supply may be measured with fair precision, but on gravelly or greatly variable soils the draft on soil moisture cannot be effectively measured. Moreover, moisture loss cannot be precisely measured on large projects or in large valleys without prohibitive expense; it can only be estimated. The rainfall may vary greatly from point to point; and it is, therefore, essential carefully to estimate percentage inaccuracy in the elements of supply which may result from rainfall variation, provided the rainfall factor is comparatively large. Possibly a still greater source of error is inflowing ground-water from neighboring high land, measurements of which are sometimes impractical.

In some of the larger valleys of the West, of which many in California, Colorado, and Arizona are typical, the inflowing ground-water is of great importance. Water from melting snows and rains in the mountain ranges may sink into the coarse textured soil of the valley rim immediately after leaving the canyons. These waters gradually move underground toward the middle or lower areas of the valley and may here form high water-tables, thus supplying water for crop production. Under such conditions, this factor may make it impractical to obtain dependable measurements of the project or valley water supply.

In general, the measurement of surface run-off from the farm is neither difficult nor very expensive. Likewise, the measurement of project run-off depends on the particular project. On some projects situated as long strips of side-hill land, it is difficult to measure run-off. The measurement of run-off from valleys, in general, is not so difficult as it is from projects. Some valleys have deep canyon drainage outlets over impervious floors, and in such cases the run-off can be measured without serious difficulty. Some valleys have surface-flow and ground-water flow outlet drainage, but if it is evident that the surface flow greatly predominates, the measurement may permit a close approximation of valley consumptive use.

That the dependability of measurement of the consumptive use rests on a complete analysis of all these factors is not likely to be over-emphasized. Moreover, reports of consumptive use, for the farm, for the project, or for the valley, or, indeed, for any other area, should not be given great weight, unless they are accompanied by complete and detailed descriptions of the conditions under which they were made. Doubtless, in some localities, the relative magnitudes of the inaccessible inflow and outflow below the ground surface, or those under other conditions which render measurements impracticable, are so great as to make computations of project or valley consumptive use of little value if not really dangerous. On the contrary, the need for economic utilization of water from all sources and the urgency of being able to predict with a fair degree of accuracy the areas that may be adequately served by a given water supply seem fully to justify extraordinary efforts toward increasing the ability to measure consumptive use under specified conditions.

Experimental Measurements

The literature on duty of water is voluminous. The Federal Department of Agriculture, through its Bureaus of Public Roads and of Plant Industry; the Department of the Interior, through the Bureau of Reclamation and the Geological Survey; the State Agricultural Experiment Stations; the State Engineers' Offices; and some private agencies are annually contributing substantial amounts of time and energy to a solution of the duty-of-water problem. In Canada, also, very careful study is being made of duty of water by public and private agencies. Some of the experiments thus conducted give a fair basis for determinations of the consumptive use and the farm consumptive use. Many duty-of-water studies have been conducted without measuring the water obtained from capillary soil moisture, and, indeed, some have failed to report the crop-year rainfall. Consequently, reference is here made only to those experiments which seem most directly to contribute toward a determination of consumptive use both in its basic sense, *U*, and in its modified meanings, for the farm, the project, and the valley, respectively.

Consumptive Use, *U*, as Determined from Tank or Pot Experiments

Hammatt (2) and Stevens (19) have endeavored to determine the consumptive use from the crop yield on a given area and estimates of the evapo-transpiration ratio. Most of the evapo-transpiration studies have been conducted in tanks or potometers. Kiesselbach (10) found that the limitation of the amount of soil used, because of the small capacity of the potometer, may be a great source of error in pot experiments. It seriously affects not only the transpiration relationships, but the entire development of the plant.

Other difficulties in determination of consumptive use by means of potometers are:

(*a*) Obtaining the same atmospheric environmental conditions around pots as exist in ordinary cropped land.
(*b*) Obtaining the same number of plants per unit area of land as occur under field conditions.
(*c*) Getting the soil into tanks in the same physical conditions as exist in the field, thus permitting the same aeration and water movement.

The influence of these factors on the consumptive use is great enough to render the pot method of determining such use of questionable value.

Consumptive Use as Determined on Field Experimental Plots

Direct measurements of consumptive use in field plots are believed to be more dependable than measurements with pots or tanks. In order to permit measurements in the field, it is clearly necessary to use land in which the water-table is at a considerable distance below the surface, because when working with small field plots it is usually impractical to measure the quantity of water absorbed by the crop from the ground-water. It is also necessary that the irrigation water be applied in small units not to exceed a depth of 5 in. in a single irrigation on ordinary soils. Even a smaller quantity will often result in percolation beyond the depth of feeding roots in coarse-textured soils. Either very large unit applications of water at ordinary time intervals, or excessively frequent irrigations resulting in the maintenance of a high moisture content, is likely to cause an appreciable downward movement of moisture in the gravitational or capillary form, and thus give apparent values for the consumptive use higher than the true values. It is impracticable, by direct means, to measure the amount of such downward movement, and hence an attempt to measure the consumptive use under these conditions may give, in reality, the farm consumptive use.

When the percolate equals zero, the farm consumptive use equals the consumptive use; therefore, it follows that the consumptive use equals the supply less the surface run-off. In most of the field determinations of net duty of water, the run-off has been either carefully measured or reduced to zero by the proper preparation of the experimental plots. The supply factors, in general, have not been measured with precision even on plots where the great depth of the water-table has precluded the possibility of the crop receiving any ground-water. This is due to the fact that precise measurements of the quantity of water taken from capillary soil-moisture storage is practical only in comparatively homogeneous soils free from gravel, and that even under the most favorable soil conditions dependable soil moisture tests add greatly to the cost of duty-of-water studies. There is, however, a growing need for more thorough duty-of-water studies in which the consumptive use will be determined for different conditions. In some such studies the consumptive use has been either determined or closely approximated, and of these the following are typical.

Widtsoe's Utah Work.—Widtsoe (23, 24) pioneered the measurement of consumptive use in field plots at the Utah Agricultural Experiment Station, beginning in 1902. This work was done on land having a water-table about 75 ft. below the surface; and, hence, it is reasonably safe to conclude that the crops obtained no ground-water and that the crop-season rainfall, the draft on stored capillary soil moisture, and the irrigation water furnished all the water to which the crops had access. There was no run-off from the experimental plots used by Widtsoe, $R_f = 0$. The deep percolation losses, D_f, were not measured. If these losses are assumed to be negligible, then, $U = H_f$, that is, the sum of the quantities, (*a*), (*b*), and (*c*), of H_f as used in Equation (3). Widtsoe measured these sources of water for 14 crops during the 10-year period, 1902-11, inclusive. Results for the 7 crops on which most of the work was done, are given herein. The crop-season rainfall used by Widtsoe was 0.42 ft. and the seasonal draft on capillary moisture in the upper 8 ft. of soil, varied from 0.10 ft. for corn to 0.83 ft. for alfalfa. Quantities of irrigation water were applied, varying from 0.42 ft. to 5.00 ft., and wide variations in crop yields were obtained. In the studies by the Committee the yields obtained by Widtsoe were plotted against the total water used, and, as a basis for arriving at the consumptive use, those yields were selected which appear to be most profitable. With nearly every crop, the yield increased rapidly to a certain point with increase of total water used, and then either decreased with further increase in water or increased very slowly, and at this "break in the curve", the value of U was taken. The results are given in Table 1.

TABLE 1.—CONSUMPTIVE USE AS DETERMINED ON FIELD PLOTS BY WIDTSOE AT THE UTAH AGRICULTURAL EXPERIMENT STATION, IN CACHE VALLEY, UTAH.

Crop.	Number of single trials.	Yield, in bushels, or tons per acre.*	Consumptive use, in acre-feet per acre.
Sugar-beets	152	21.8	2.5
Potatoes	124	267.0	2.2
Alfalfa	49	4.7	3.3
Corn	81	99.2	2.5
Wheat	142	45.7	2.4
Oats	29	80.7	2.5

* Yields are given in tons per acre for sugar-beets and alfalfa and bushels per acre for the remainder of the crops.

Widtsoe's work indicates the importance of yield in determining the consumptive use. It is also important to keep in mind the fact that deep percolation losses from the plots on which Widtsoe worked would result in observed magnitudes of U higher than the true ones. It is far more probable that the given values of U are too high rather than too low.

Snelson's Alberta Work.—Working on field plots in Brooks, Alberta, Canada, Snelson (18) used moderate quantities of water in single applications and made careful measurements of soil moisture to a depth of 6 ft. at the beginning and the end of the growing season. Under his methods percolation loss in all probability was very small if not zero, and hence it seems safe to assume that U is equal to U_f. If percolation losses were zero, then according to Snelson's experiments with wheat on the more fertile soil the consumptive use varied from 0.85 ft. to 1.82 ft. as the crop yield varied from 10 to 50 bushels per acre. For oats on the more fertile plots the consumptive use ranged from 0.72 ft. to 1.75 ft. as the yield varied from 40 to 135 bushels per acre. Barley required a consumptive use from 1.25 to 1.60 ft. for yields ranging from 40 to 51 bushels per acre, and for alfalfa the use varied from 1.00 to 2.62 ft. for yields ranging from 1.0 to 5.7 tons per acre.

Powers' Oregon Work.—Powers (16, 17) has made many field plot measurements of consumptive use in Oregon. Experimenting with alfalfa in the Willamette Valley during 1911, using moderate irrigation he found values from 1.4 to 2.0 ft., accompanying yields of 4.1 to 5.2 tons per acre. The consumptive use for clover was approximately the same as that for alfalfa. Moisture determinations were made to a depth of 6 ft. at the outset in the Willamette Valley work, but as most of the borings showed a water penetration to only 4 ft., the later borings were not made below this depth except in connection with a few very heavy irrigations. Powers was convinced that there was "practically no loss by percolation" apparently because dry soil was frequently encountered in the lower depths and also because these lower depths in many instances gained in moisture content during the period following one irrigation and preceding the next following. It is doubtful whether this is conclusive since the increase in the lower depths may have been caused by a movement of capillary water downward from the moist surface to the dryer sections below. If such were the case, some percolation probably occurred and Powers' measurements would represent U_f instead of U. If, on the other hand, the moisture did rise in significant quantities from below the zone of soil sampling, then the value of the consumptive use as measured by Powers is too small as there was an unmeasured source of water. In reporting the Western Oregon work, Powers is silent concerning the depth of the water-table.

Further measurements were made in Eastern Oregon by Powers (17), but it is beyond the scope of this report to give them in detail.

Utah Work by Harris.—Further field plot work at the Utah Agricultural Experiment Station by Harris (6) and associates from 1911 to 1919 gives additional information concerning consumptive use. The results of this work, together with the work done by Widtsoe, have been reported in detail in fifteen Station *Bulletins* which were published as the work progressed. Finally, in one paper (6), Harris has brought together the results of seventeen years' work. This *Bulletin* contains six charts, each representing a different crop, which show the relation between the yield and the quantity of water applied. Two curves are platted on each chart, the actual average yields for the different irrigations being shown by dotted lines, and the average found by weighting the results according to the number of tests is shown by heavy lines. Harris did not report the seasonal draft of each crop on the soil moisture. For purposes of comparison, this has been assumed to be the same as that found by Widtsoe. Moreover, to make the results still further comparable, the depths of water applied, according to the heavy lines of the charts prepared by Harris, have been selected corresponding to the respective crop yields used in connection with Widtsoe's data, that is, for 21.3 tons of sugar-beets, 267.0 bushels of potatoes, etc. On these bases, Table 2 has been prepared.

The values given in this table show fair agreement with those obtained from Widtsoe's work, except for the alfalfa and oats. According to Fig. 5 of Utah Station *Bulletin 173,* a yield of alfalfa of 4.3 tons per acre would require the same consumptive use as found by Widtsoe for 4.7 tons, namely, 3.3 ft. The maximum yield of oats according to the 17 years' work reported by Harris is slightly less than the 80.7 bushels per acre used in Widtsoe's work. A yield of 74 bushels per acre, according to the curves by Harris, would have been produced by a consumptive use of 2.5 ft., the quantity required to produce 80.7 bushels in Widtsoe's work.

TABLE 2.—CONSUMPTIVE USE APPROXIMATIONS BASED ON THE FIELD PLOT WORK OF WIDTSOE AND HARRIS AT THE UTAH AGRICULTURAL EXPERIMENT STATION.

Crops.	Yield, in tons or bushels per acre.	Depth of water applied, in feet.	Depth of soil moisture and rainfall used, in feet.	Consumptive use, in acre-feet per acre.
Sugar-beets	21.3	1.50	0.86	2.4
Potatoes	207.0	1.67	0.52	2.2
Alfalfa	4.7	3.33	1.25	4.6
Corn	99.2	1.67	0.52	2.2
Wheat	45.7	1.33	1.15	2.5
Oats	80.7	2.33	0.81	3.1

FARM CONSUMPTIVE USE

Lewis' Idaho Work.—Lewis (11) measured the quantity of water used by nine important crops in the Snake River Valley, near Twin Falls, Idaho. He divided his observations into two groups: (1) Those on the better plots that produced high yields; and (2) those on which average yields were produced. His data are summarized in Table 3. It will be noted that in the better plots the seven grain crops and also the potatoes consumed less than 2 acre-ft. per acre. The alfalfa consumed the largest quantity and the clover the next largest, the average for all the crops being 1.75 ft. on the better plots and 1.53 ft. on the average plots. It is probable that these magnitudes represent the farm consumptive use, U_f, for the Twin Falls area more nearly than the consumptive use, U, since the experimental conditions very likely permitted some deep percolation.

Hemphill's Colorado Work.—Working in the Cache la Poudre Valley, Colorado, during 1916 and 1917, Hemphill (8) measured the water applied to twenty-five farms. The run-off, R_f, is not given for each farm, but the average, R_f, was 6% of the quantity applied. Deducting 6% of the quantity for maximum crops to arrive at the quantity of irrigation water held on each farm and adding arbitrarily an allowance of 0.3 ft. for moisture absorbed from the stored winter precipitation and adding, further, 0.92 ft. for rainfall from April 1 to September 30, there results the farm consumptive use as given in Table 4. How closely this use, as given in the table, approaches the consumptive use is conjectural, as the magnitude of the percolation losses is not known and as it is subject to greater variation in actual farm irrigation than in field plot experiments heretofore reported.

TABLE 3.—FARM CONSUMPTIVE USE (U_f) AS MEASURED ON FIELD PLOTS IN THE SNAKE RIVER VALLEY, IDAHO, BY LEWIS.

Crop.	U_f on plots of high production.	U_f on plots of average production.
Wheat	1.78	1.18
Oats	1.84	1.45
Barley	1.68	1.58
Peas	1.66	1.36
Beans	1.20	1.20
Corn	0.96	1.29
Potatoes	1.63	1.60
Clover	2.14	1.54
Alfalfa	2.89	2.55
Mean, if crops are given equal weights	1.75	1.53

Investigations of the net duty of water in Sevier Valley, Utah, by Israelsen and Winsor (9) during the 7-year period, 1914 to 1920, gives some information concerning farm consumptive use. In these experiments, the quantities of water found necessary to deliver to the farm for sugar-beets, potatoes, and alfalfa, excluding farm run-off, R_f, were 2.5, 2.0, and 2.8 acre-ft. per acre, respectively. The crop-season rainfall was 0.3 ft., and assuming a draft on the soil moisture of 0.3 ft., the farm consumptive use, U_f, is as follows: Sugar-beets, 3.1; potatoes, 2.6; and alfalfa, 3.4 acre-ft. per acre. Further reference is made to these quantities in connection with a report for U_v for the Sevier Valley.

TABLE 4.—FARM CONSUMPTIVE USE AS DETERMINED BY HEMPHILL IN CACHE LA POUDRE VALLEY, COLORADO.

Crops.	Maximum yield, in tons, or bushels per acre.	Farm consumptive use for maximum yield per acre.	Average yield, in tons, or bushels per acre.	Farm consumptive use for average yield per acre.
Sugar-beets	15.1	3.6	12.6	3.0
Potatoes	390.0	4.2	230.1	3.3
Alfalfa	4.0	5.8	2.8	3.7
Barley	60.0	3.4	40.7	2.3
Wheat	45.0	3.1	27.7	2.1
Oats	80.0	3.6	48.1	1.4

PROJECT CONSUMPTIVE USE

It is important to remember that farm and project consumptive use in general are likely to be, respectively, equal to or greater than the consumptive use, and also that during the early years of irrigation the difference between these quantities is likely to be greater than after many years of farming under the canal. The following are examples of some approximations of project consumptive use.

Steward's Idaho Work.—In the Boise Valley, Idaho, in connection with drainage operations, Steward (20) found project consumptive use on the Pioneer District Drainage area (approximately 13 500 acres) by subtracting the drainage run-off from the sum of the rainfall plus the irrigation water applied, a difference of 2.13 acre-ft. in 1916 and 2.18 acre-ft. in 1917. On the Twin Falls South Side Project, in 1915, Sloan (20) found, by working on a 12 000-acre tract, a project consumptive use of 2.95 acre-ft.

Steward and Paul (21) made a careful study of sources and disposition of water on the Nampa Meridian Drainage Districts in 1917. These investigators found on the Mason and Indian Creek tracts, having an area of approximately 50 000 acres, a transpiration plus evaporation of 1.9 ft. from April to October, inclusive. The factors of supply, H_p, of Equation (5) were as follows: (*a*) The inflow consisted largely of irrigation water and was 4.16 acre-ft. per acre; (*b*) the draft on soil moisture was neglected, probably without greatly influencing the results; (*c*) the absorption from the ground-water was not estimated; and (*d*) the crop-season rainfall was 0.36 ft. The factors in project run-off of Equation (5) were measured as follows: (*a*) Run-off was measured at all drainage outlets, and amounted to 2.02 acre-ft. per acre; (*b*) the deep-soil percolation losses were measured by making direct observations on water-table fluctuations and amounted to 0.60 acre-ft. per acre; and (*c*) the deep-soil percolation losses which re-appeared at the boundaries of the project, were measured as waters commingled with those in Factor (*a*). Applying Equation (5), the approximate project consumptive use, $U_p = (4.16 + 0.36) - (2.02 + 0.60) = 1.90$. A complete report of the work of Steward and Paul is beyond the scope of this report.

Crandall's Snake River Valley Work.—Crandall (1) measured consumptive use in the Lower Snake River Valley. He considered an irrigated area of nearly 90 000 acres in 1917 and of more than 100 000 acres in 1918. Using three different methods, he obtained close agreement. These methods were:

1.—Measurement of percolation losses from the irrigated land by finding the annual increase in discharge of springs below the tract.
2.—By making soil moisture determinations before and after irrigations.
3.—By computations from water-requirement studies under somewhat similar climatic and soil conditions.

A brief review of each method and of the results obtained are given as follows.

1.—Crandall's Measurement of Percolation Losses

The normal annual discharge of the springs as determined by measurements for a period of 5 years prior to irrigation was 3 885 sec.-ft. By allowing a 2-month lag period—the time required for the seepage losses to appear in the springs—Crandall found the increases in the spring discharges from all irrigation sources and from deep percolation from farms, and knowing the total deliveries to irrigators, he determined the consumptive use as shown further in Table 5.

Crandall believes that delay in water deliveries until the middle of May in 1917, due to maintenance work, is the cause of the smaller consumptive use during the first year of the measurements.

2.—Crandall's Soil Moisture Measurements

By taking samples of soil before and after irrigation, Crandall found that the moisture content of the forage-crop soils increased from 10 to 20% of the dry weight of the soil and that the moisture content of the grain-crop soils increased from 8 to 20%—each to a depth of 4 ft. The water-free soil weighed 80 lb. per cu. ft.; hence, the upper 4 ft. of the forage-crop soils absorbed 0.5 ft. per irrigation, or 2.5 ft. for the five irrigations; the same depth of the grain-crop soils absorbed 1.8 ft. from three irrigations, or 0.6 ft. in each irrigation. About 54% of the project was in forage crops and 45% in grain crops; hence, the consumption of irrigation water on this basis was 2.2 ft., and adding the crop-season rainfall, 0.15 ft., and the draft on soil moisture, about 0.25 ft., the consumptive use becomes 2.6 acre-ft. per acre.

TABLE 5.—PROJECT CONSUMPTIVE USE AS DETERMINED BY CRANDALL ON THE TWIN FALLS NORTH SIDE PROJECT, SNAKE RIVER VALLEY, IDAHO.

		From June, 1917, to May, 1918, included, in acre-feet.	From June, 1918, to May, 1919, included, in acre-feet.
Increase in springs discharge from all irrigation sources		816 800	878 900
Losses from canal system by seepage plus spring water contributions from other irrigation projects		612 627	650 439
Contributions to spring discharge by seepage from irrigated lands		204 173	228 461
Deliveries to irrigators less run-off or recovered wastes		389 909	484 838
Irrigation water consumed by the crops	Total on the project	185 736	256 377
	Acre-feet per acre	2.1	2.5
Rainfall during crop season, April to September, inclusive		0.15	0.15
Draft on moisture in soil		0.25	0.25
Project consumptive use, U_p		2.5	2.9

3.—Crandall's Computations on Basis of Water-Requirement Studies

At the beginning of this Committee's consideration of experimental observations, the difficulties and common sources of error in the use of water requirement studies from pot or tank experiments as a basis for consumptive use determinations over large areas were briefly enumerated. That determinations of the consumptive use on this basis could be considered only as approximations was also asserted. Despite these facts, it is interesting to note the agreement obtained by Crandall using this method with the two methods just presented. The analysis made by him is quoted herewith from his report:

"Detailed experiments have been made by Widtsoe at Logan, Utah, under climatic and irrigation conditions similar to those existing on the North Side tract, to determine the amount of water required to produce a pound of dried crop. The amount of water so required varies with the different factors including kind of crops, degree of fertility, amount of water applied, amount of soil moisture at beginning of season, etc. Detailed results are published in *Bulletin 285* of the U. S. Department of Agriculture and *Bulletin 116* of the Utah Experiment Station. For the conditions existing on the North Side Tract during 1917, these experiments indicate that approximately 1 300 lb. of water would be required to produce a pound of dried crop. This includes the transpiration requirements of the plants and evaporation losses from the fields. The average yield of the hay and pasture crops during 1917 was 4½ tons per acre. As the hay when cured in the stack contains about 15% moisture, this is equivalent to about 7 600 lb. of dry matter per acre for the hay and pasture crops. The average yield of wheat was determined as approximately 25 bushels per acre and the straw as 1½ times the weight of the grain making a total production of 3 800 lb. of crop per acre for wheat. As the wheat and straw contain about 10% moisture, this is equivalent to 3 400 lb. of dry matter per acre, which is taken as the average yield for crops aside from hay and pasture. The low average grain yields were due to the large acreage on which grain was planted late in the spring as a nurse crop for alfalfa.

"The average yield over the entire tract for all crops was then approximately 5 600 lb. of dry matter per acre. At an average water cost of 1 300 lb. of water per pound of dry matter this would require for its production 2.65 acre-ft. of water per acre. This quantity includes the water withdrawn from the moisture stored in the soil and precipitation during the growing season."

The water requirement, 2.65 acre-ft. per acre, is equivalent to the consumptive use as herein defined. It must be noted, however, that percentages of moisture in the hay and in the grain are estimates only, that the probable error in measuring the crop yields is unknown, that the percentage of wheat and straw likewise is estimated, and that in the light of these conditions the close agreement of the determinations of the consumptive use by the several methods should not be considered as conclusive evidence of the dependability of the method.

Valley Consumptive Use

In the early years of irrigation of any valley, the valley consumptive use is relatively large because the deep percolation losses, D_v, go to moisten dry soil to great depths and build up the water-table, and are, therefore, temporarily non-recoverable. Moreover, the non-cropped area is relatively large during the early years of irrigation, and losses therefrom are larger than in later years.

Sevier Valley, Utah.—The Sevier Valley, in Utah, is typical of the older irrigated valleys in which deep percolation losses have become very small, since nearly all these losses shortly reach the high water-table which slopes toward the stream channels (or is quickly built up and thus made to slope toward the channels) so that there are very small, if any, non-recoverable losses from percolation. Likewise, the non-cropped area is small, and hence its losses are practically negligible. In this valley it is very probable that the consumptive use is almost equal to valley consumptive use; hence, from Equation (7), $U = H_v - R_v$. The inflow to, and the outflow from, the Sevier Valley for the 7-year period, 1914 to 1920, has been measured by the Utah State Engineer's Office and reported by Ullrich (22). The average annual inflow of the river at Sevier, including the inflow of tributaries from Sevier to Gunnison, for this 7-year period was 295 000 acre-ft., and the average outflow was 211 000 acre-ft., thus leaving 85 000 acre-ft. which was applied to 65 000 acres giving a consumptive use of 1.3 ft. The crop-year rainfall is approximately 0.3 ft. As the mean annual rainfall is only 0.7 ft., the draft by the crops on moisture stored in the soil from winter rains and from ground-water is probably not to exceed 0.3 ft., the quantity previously assumed in connection with the net-duty work. On this basis, therefore, the consumptive use, U, for the Sevier Valley, as herein defined, is 1.9 ft. This value of the consumptive use is much less than farm consumptive use as determined by the Utah Experiment Station for standard crops on a typical soil.

If one-half the irrigated lands of the valley were in alfalfa and one-fourth in sugar-beets and potatoes, respectively, then farm consumptive use, according to the values presented heretofore, would be 3.0 acre-ft. per acre. For crops thus distributed and for yields equal to those of the experimental farm it would mean either that farm percolation $D_f = 1.1$ acre-ft. per acre, since $U_f = U + D_f$ (provided no water entered the valley through underground sources that could not be measured), or that 71 500 acre-ft. percolated into the valley if $D_f = 0$. In all probability the truth lies somewhere between these two extremes, meaning that some unmeasured water entered the valley and that some water was lost by deep percolation from the experimental plots, the sum of the two for the entire valley being approximately 71 500 acre-ft., provided the experimental farm yields were not much greater than those of the valley as a whole.

Cache la Poudre Valley, Colorado.—The Cache la Poudre Valley, in Northern Colorado, is one of the foremost irrigated valleys of that State. Irrigation practice there is diversified, well-balanced as to crop rotation, and economic as to use of water. The need for water has raised water values and forced economy of use in irrigation. A 2-year investigation of irrigation in that area furnishes some valuable information on consumptive use. According to Hemphill the average consumptive use of river water, during the 2-year period of 1916-17, was 1.25 acre-ft. per acre over an area of 220 000 acres net. Adding 0.92 acre-ft. crop-season rainfall which is 80% of the 1.14 acre-ft. mean annual rainfall, gives a valley consumptive use of about 2.17 acre-ft. per acre. Data are not available concerning draft on soil moisture.

Truckee River, Nevada.—In connection with duty-of-water studies on the Truckee River, in Nevada, Harding (3) estimated the valley consumptive use by subtracting the valley outflow at Vista from the inflow at the California-Nevada line. He made estimates also based on direct measurement of drains and found fair agreement for seasonal consumptive use between the two methods. In making monthly comparisons by the two methods the results are less consistent. The gross area of cropped land considered was 27 000 acres, approximately three-fourths of which is cultivated land and the remainder largely wild hay land. The cultivated land produced mostly alfalfa, but also some native grass on land that might have been used for other crops. Based on these studies, Harding concludes that the valley consumptive use during the six months of April to September, inclusive, for 1900 to 1918, including the 0.19 ft. rainfall, is 3 ft., but excluding the draft on soil moisture, if any. The local practice of applying water at frequent intervals in small quantities is considered in part responsible for this high consumptive use. Moreover, the meadow areas are irrigated almost continuously. The irrigation season is about 165 days. A high precision is not claimed for the results, as evidenced in the following comments by Harding:

> "The records are not as complete as would be desired, and the resulting consumptive use is higher than would be expected. The area is typical of two cuttings of alfalfa with either a light third cutting or good fall pasturage valley such as occur in the Mountain States."

Columbia River Tributaries.—Parker (14) estimated the average valley consumptive use, exclusive of rainfall, on 12 tributaries of the Columbia River for the "climatic years" 1909-10, 1910-11, and 1911-12. The estimated irrigated acreages during each of the three years were 99 000, 103 000, and 108 000, respectively. It is noteworthy that the magnitudes of consumptive use obtained, namely, 2.9, 2.7, and 2.8, respectively, agree closely, despite significant differences in the run-off.

METHODS OF ESTIMATING VALLEY CONSUMPTIVE USE

Hedke's Method.—It is desirable to estimate the consumptive use of water in irrigation, in different valleys, in advance of complete agricultural development. Recognizing the importance and complexity of the consumptive use problem, Charles R. Hedke, M. Am. Soc. C. E., (7) has investigated the relation of consumptive use of water to the quantity of heat available to the crop during the growing season, and has found substantially that under favorable agricultural conditions the use of water is directly proportional to the use of heat available. The relative approach of the actual agricultural practice to the best practice is designated by Hedke as the "Standard of Agriculture". The application of the direct relation between water consumption and available heat, proposed by Hedke, to a valley in which the agricultural practices are of a high standard, necessitates the following assumptions:

1.—That the heat consumed by a particular crop, during any day or other time period, is determined by the amount of heat available to the crop above the germinating or minimum growing temperature.

2.—That under favorable agricultural practices, each crop consumes water in direct relation to the heat available as defined.

3.—That the soils considered are abundantly supplied with moisture and plant-food so that the yield of a crop will be limited only by the amount of heat available.

4.—That the influence of variations in wind velocity, relative humidity, and vapor pressure on consumptive use of water are relatively small as compared to the influence of available heat.

Under these assumptions, Equation (1*a*) may be written, $U = f\,(Q_h)$, or simply $U = K\,Q_h$, which is the basic relation assumed by Hedke.

To determine Q_h, Hedke subtracts the minimum growing temperature of each crop from the mean temperature of each month, or fraction thereof, within the growing season. The result thus obtained, multiplied by the number of days in the month, represents the available heat units in day-degrees for a particular crop during the month considered. The sum of the heat units available to each crop for each month gives the available heat units for the crop-year. Hedke found in the Cache la Poudre Valley of Colorado in 1916 that heat was available to alfalfa during the eight months of March to October, whereas, to potatoes it was available only from May to September, inclusive. To find the total heat units available on a project, the heat units available to each of the crops are multiplied by the proportion of the project lands in the respective crops, and the sum of the products thus obtained gives the available heat units for the project.

Hedke has applied his proposed principle to several typical valleys and has obtained some gratifying results. The result of his work in the Cache la Poudre Valley and its application to the San Luis Valley, in Colorado, and the Rio Grande Valley, in New Mexico, suggests that Q_h is a predominating factor in fixing the magnitude of the consumptive use.

Hedke defines consumptive use as the "total amount of water transpired incidental to farming an area"; thus the inclusion of the water evaporated from the crop-producing land is implied, but not specifically stated. He states further that the consumptive use "is here covered by the net depletion from the stream plus one-half the total yearly rainfall". However, his applications do not show that "net depletion" includes the crop-year draft on soil moisture plus the water derived from the ground-water, both of which should be considered as indicated in this Committee's enumeration of the elements of supply, as used in Equation (7).

Based on Hemphill's measurements of valley consumptive use for the Cache la Poudre Valley, and using the equation, $U_v = K\,Q_h$, Hedke has found that $K = 0.000423$, approximately. Having the magnitude of K, Hedke considers it possible to determine the valley consumptive use for the highest "standard of agriculture" in any valley in which dependable crop distribution and temperature records are available. His analysis of crops and temperatures in the Middle Rio Grande Valley shows a normal available heat, Q_h, of 6 200 day-degrees, and applying the value of K, finds, $U_v = 0.000423 \times 6\,200 = 2.6$ acre-ft. Considering the Rio Grande Project, he finds that $Q_h = 6\,800$, and, hence, $U_v = 0.000423 \times 6\,800 = 2.9$ acre-ft. By deducting the "effective precipitation" from each of these values, he arrives at an estimate of the net depletion in the stream system. The application of these estimates without modification is of necessity based on an actual or an assumed high "standard of agriculture" in the Rio Grande Valley and on the Rio Grande Project similar to that of the Cache la Poudre Valley of Colorado.

In this connection it is essential to remember that Hedke urges the importance of making careful observations concerning the relative agricultural practices in different valleys for which information is available, concerning the total available heat termed by him the "thermal location", and that the magnitude of U_v, as computed from a known constant and the available heat, be reduced in proportion to the lack of attainment of the highest possible standard of agriculture. The dependability of such reduction in the computed valley consumptive use obviously rests on the experience and judgment of the engineer. The method proposed by Hedke, therefore, seems to give the greatest promise in those valleys where a high standard of agriculture has been attained, thus eliminating the necessity of applying the so-called "personal equation of judgment and experience". Obviously, the method may likewise be applied to the newer irrigated valleys under an assumed ultimate distribution of crops and high agricultural standard for the purpose of estimating the consumptive use of water which will occur after the agricultural development of the valley concerned has been completed.

Harding's Method.—Harding has studied project and valley consumptive use on extensive areas in the San Joaquin Valley, California (4, 5). A complete report of his California investigations is beyond the scope of this report, which is restricted to a brief statement of the extent of the work, a description of his methods, and a summary of his observations.

The Kings River area included about 700 000 acres of irrigated land and the Kaweah area, 184 000 acres. Harding measured the factors of water supply and run-off of Equation (5), as follows:

(*a*) The channel inflow is based on records of current-meter rating stations.
(*b*) The crop seasonal draft on soil moisture was not measured; in effect, however, it was reduced to a minimum by so modifying other factors as to provide an equivalent elevation of water-table.
(*c*) The necessity of computing the quantity of water absorbed from the ground-water was eliminated by the same method as the moisture content in Factor (*b*).
(*d*) The crop-seasonal rainfall on the projects investigated is negligible.
(*e*) There were no appreciable surface run-off losses.
(*f*) The need for measurements of deep percolation losses was eliminated by determining graphically the quantity of canal water needed to maintain the water-table at a constant normal elevation (see Fig. 1).

A similar statement applies to Harding's method of determining H_v and R_v in Equation (7).

Harding's Method as Applied to the Kings River Area.—The conditions in some of the areas served by the Kings River (4) are such as to furnish a basis on which the consumptive use may be estimated. These conditions are most favorable in the Fresno, Consolidated, and Alta Irrigation Districts. For these districts adequate records of ground-water fluctuation were available for 1922 to 1925, which include one year of more than normal run-off, one year of about 80%, one of about 70%, and one of about 25% run-off. The relative diversions vary even more widely than the run-off due to the later priorities of rights of the Consolidated and Alta Districts. Adequate diversion records are also available for the districts as a whole and for their main laterals.

In connection with the engineering studies made by the State Engineer's Office of the Kings River Water Conservation District, Harding made a detailed analysis of the ground-water records.

The rainfall at Fresno averages 0.81 ft. annually, and occurs mainly in the winter months. Diversions for irrigation extend from February to October, varying with the available water supply, the period of diversion being longer in the Fresno District due to its earlier priority of right.

The method of analysis consisted in determining the seasonal ground-water fluctuation and comparing it with the acre-feet of water conveyed into the area, per acre of cropped land. In all these areas there is extensive pumping for irrigation. Some land was supplied with water entirely by pumping from local ground-water; other land was furnished canal water when available and pumped water during the remainder of the season. The gross area of farms served was used in estimating consumptive use. As the projects are highly developed, the net area cropped is nearly as large as the gross areas herein used.

The year was divided into two periods, namely, March 1 to November 30 and December 1 to February 28, because an examination of the ground-water fluctuations in the winter indicated no influence from the delivery of water during the preceding irrigation season. The winter fluctuations vary largely with the winter precipitation, but this is probably an indirect relationship, since with larger rainfall there is less irrigation pumping, and there is also some local tributary run-off.

The canal deliveries necessary to maintain the ground-water were taken as the quantities necessary to cause a rise or fall from March 1 to November 30 equal to the fall or rise that would occur from December 1 to February 28 in a year of normal rainfall. The quantities of water delivered to each tract each year, together with the accompanying water-table fluctuations, were plotted as illustrated in Fig. 1, which typifies Harding's method of analyzing consumptive use data. This diagram gives the results obtained, respectively, on the entire area under the Fresno Irrigation District, the Consolidated Irrigation District, and the Alta Irrigation District.

The summarized results of canal deliveries and ground-water fluctuations for these three areas for 1922 to 1925 are given in Table 6. The resulting estimated requirements for diversion into each area to supply the consumptive use and maintain the ground-water are also given in Table 6. These estimates were arrived at by plotting the ground-water fluctuation against the deliveries, as shown in Fig. 1, and selecting the intersections of the line drawn through the resulting points with the axis of zero change in level of ground-water, as representing the deliveries that would be required to supply the consumptive use and maintain the ground-water for the periods, March 1 to December 1. The estimated total annual requirements were based on allowing for the ground-water fluctuations from December 1 to March 1 in years of normal rainfall as previously explained.

The indicated consumptive use as obtained from Fig. 1 and Table 6, is based on the surface diversions into the areas. It does not include any unmeasurable ground-water movement into or out of the area. Ground-water inflow is considered to be relatively small as the areas above these districts are unirrigated and the adjacent Kings River channel shows a gain rather than a loss under normal conditions of supply. Ground-water outflow may occur, although its extent is considered to be relatively small in proportion to the consumptive use so indicated by other records.

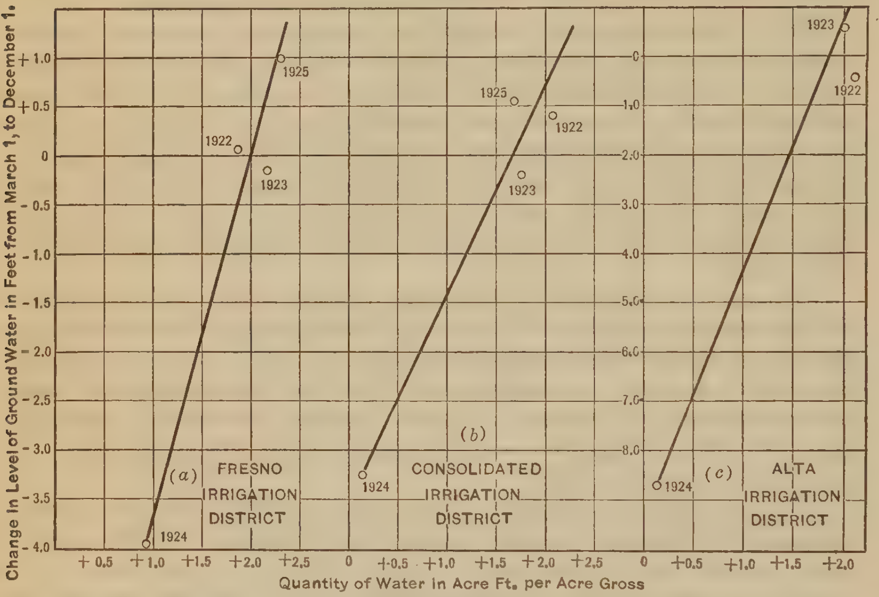

FIG. 1.—RELATION OF VOLUME OF WATER DIVERTED IN AREA TO CHANGE IN LEVEL OF GROUND-WATER.

In 1922 and 1923, in some parts of each of these three districts the ground-water was within 6 ft. or less of the ground surface. In some areas it rose within less than 4 ft. of the surface. The estimated delivery requirement to maintain the crops and ground-water includes evaporation from the soils in such areas, as well as plant transpiration. The consumptive use indicated may be affected by these conditions. The numerical results presented are considered as illustrating the method used rather than as a final determination of the quantity of the consumptive use of these areas. A final determination will require additional length of record under present ground-water conditions.

In addition to the areas of the district as a whole, the areas served by several of the main laterals were similarly studied. Such areas had varying conditions as to the type of crop grown and ground-water elevations. From these results the following conclusions were drawn by Harding:

"Records to date from different areas indicate, as tentative conclusions, that in areas of trees and vines having limited ground-water outflow, the crop needs for moisture will be supplied by an average annual delivery into the area of about 1⅔ acre-ft. of water per acre of crop. For forage crops an average delivery of 2 to 2¼ acre-ft. of water per acre of crop may be required. For other crop conditions other amounts of supply are found necessary. Where ground-water outflow occurs or where the ground-water is close to the ground surface, larger amounts are required to supply the crops and maintain the ground-water."

TABLE 6.—DIVERSION OF WATER AND GROUND-WATER FLUCTUATIONS IN FRESNO, CONSOLIDATED, AND ALTA IRRIGATION DISTRICTS FROM MARCH 1 TO DECEMBER 1, WITH FLUCTUATIONS FROM DECEMBER 1 TO MARCH 1 IN YEARS OF NORMAL RAINFALL.

Year.	IRRIGATION DISTRICT.					
	Fresno.*		Consolidated.		Alta.	
	Diversion into area, in acre-feet per acre irrigated, March 1 to December 1, in feet.	Average ground-water fluctuation, March 1 to December 1, in feet.	Diversion into area, in acre-feet per acre irrigated.	Average ground-water fluctuation, March 1 to December 1, in feet.	Diversion into area, in acre-feet per acre irrigated.	Average ground-water fluctuation, March 1 to December 1, in feet.
1922	1.93	+0.15	2.05	+0.45	2.07	−0.40
1923	2.12	−0.15	1.75	−0.15	2.01	+0.80
1924	0.96	−3.95	0.20	−3.15	0.18	−8.65
1925	2.30	+1.0	1.64	+0.55	1.95	+1.50
Ground-water fluctuation, December 1 to March 1, in season of normal rainfall	.....	+0.3	....	0.0	.....	+0.2
Estimated annual requirement for diversion into area to supply consumptive use and maintain ground-water, in acre-feet per acre irrigated	2.0	......	1.7	......	1.9	......
Crops in percentage of total:						
Trees	29	......	15	......	13	......
Vines	43	......	60	......	80	......
Forage and pasture	19	......	10	......	4	......
Miscellaneous	9	......	15	......	3	......

* Covers only portion of the district served by the Fresno Canal; gross area, 169 000 acres.

Kaweah River Area.—Some data indicating valley consumptive use are given by Harding (5) for the Kaweah River area during 1920 and 1921, the records indicating a valley consumptive use, U_v, of about 2.2 acre-ft. per acre. The crops consisted of approximately 34% orchard and vines, 29% alfalfa, 28% corn and grain, and 9% miscellaneous. There were some areas of high ground-water, but these were relatively small in extent. The climatic conditions were typical of those of the main San Joaquin Valley areas. The ground-water is dependent on the Kaweah River and the diversions therefrom. Available data indicate that the ground-water basin is closed. About 183 500 acres were cropped in a gross area of 365 000 acres.

TABLE 7.—CONSUMPTIVE USE OF WATER IN IRRIGATION.

River basin. (1)	State. (2)	Average elevation of land above sea level, in feet. (3)	Mean annual temperature, in degrees. (4)	Mean annual rainfall, in inches. (5)	Rainfall during growing season, in inches. (6)	Head-gate diversion, in acre-feet per acre per year. (7)	Net duty on land, in acre-feet per acre per year. (8)	Consumptive use, irrigation water, in acre-feet per acre per year. (9)	Consumptive use, including crop-year rainfall, (U_a). (9a)	Area, in acres. (10)	Growing season, average number of days between frosts. (11)	Authority and remarks. (12)
1. Little Laramie: Hatton to Two Rivers............	Wyoming..	7 300	40	14	4	(*a*)	(*a*)	0.90	1.2	28 000	95	12-year record, 1912-23, inclusive, includes some waste into Big Basin by Park Ditch; automatic gauge, Upper Station, 1912-14; automatic gauge, Lower Station, 1913-23.
2. Little Laramie: Hatton to Two Rivers............	Wyoming..	7 300	40	14	4	(*a*)	(*a*)	1.00	1.3	28 000	95	R. I. Meeker, 3-year record (1912-13-14), Laramie River investigations; automatic records of water above and below; irrigated area determined from reconnaissance survey; (*a*) chiefly meadows, heavily flooded compact area.
3. North Platte: North Park.......	Colorado...	8 200	38	10	2	(*b*)	(*b*)	1.00	1.8	120 000	60	1.00 assumed and adopted for North Platte Co-Operative Report by Conkling and Meeker (1919), see Vol. 2; (*b*) chiefly meadows; freely flooded.
4. Boise: Mason Creek Area, Boise Project, U. S. Reclamation Service...	Idaho......	....	50	11	3	4.1	......	1.20	1.4	13 550	155	U. S. Reclamation Service special investigation on area with drainage system; drainage recovery, 2.89 acre-ft. per acre per year, see North Platte Co-Operative Report, Vol. 2, p. 120 (1919); work by Steward, Paul, *et al.*
5. South Platte: Kersey to Julesburg..	Colorado...	4 100	48	14	9	2.7	1.8	1.15	1.9	229 000	145	Comp. by R. I. Meeker from special investigation by Co-operative, U. S. Department of Agriculture, Colorado Agricultural Experiment Station, and State Engineer of Colorado, 1919-20, Colorado Agricultural Experiment Station, *Bulletin 279* (December, 1922); water-table still rising and return flow increasing; includes reservoir evaporation.

TABLE 7.—(*Continued.*)

River basin. (1)	State. (2)	Average elevation of land above sea level, in feet. (3)	Mean annual temperature, in degrees. (4)	Mean annual rainfall, in inches. (5)	Rainfall during growing season, in inches. (6)	Head-gate diversion, in acre-feet per acre per year. (7)	Net duty on land, in acre-feet per acre per year. (8)	Consumptive use irrigation water, in acre-feet per acre per year. (9)	Consumptive use, including crop-year rainfall, (U_v). (9 a)	Area, in acres. (10)	Growing season, average number days between frosts. (11)	Authority and remarks. (12)
6. Cache la Poudre: Poudre Valley....	Colorado...	5 000	48	14	7	1.9	1.5	1.25	1.8	220 000	135	Special 2-year investigation by Hemphill of the U. S. Dept. of Agriculture, 1916-17, *Bulletin 1026* (1922), pp. 24, 25; footnote on page 25 states credit for return seepage from Poudre Valley Land into South Platte would reduce slightly; the 1.25 figure determined includes reservoir evaporation.
7. South Platte: South Platte and Tributary Valleys..............	Colorado...	5 700- 3 500	49	14	7	2.0±	......	1.25±	1.8	1 100 000	145	Water supply study and analysis for 10-year period, by R. I. Meeker, 1912-21. (See *Engineering News-Record*, July 20, 1922, pp. 105-109). Water-table still rising and return flow increasing; future consumptive use will be less than 1.25 acre-ft. per year; includes evaporation losses from 300 reservoirs.
8. Sevier: Sevier to Gunnison...............	Utah	5 200	48	8	2	......	......	1.30	1.5	65 000	110	Exhibit A, Appendix, Salt Lake City Hearings, Colorado River Commission, 1922; C. J. Ullrich, eight-year record; 1915-22.
9. Boise: Part Boise Project, U. S. Reclamation Service...	Idaho	2 500±	50	11	3	......	3.6	1.30	1.6	49 090	155	U. S. Reclamation Service Special Investigation on area with drainage system; drainage recovery, 2.29 acre-ft. per acre per year; 3-year period; see North Platte Co-Operative Report, Vol. 2 (1919), p. 120; excludes seepage from Deer Flat Reservoir.

Meeker's Valley Consumptive Use Studies in Colorado.—Meeker has conducted investigations concerning consumptive use in several typical valleys in Colorado and has also assembled and compiled data for other valleys. His analyses were made with the viewpoint of determining the net depletion of river water by irrigated lands either naturally or artificially drained. Wasteful evaporation from seeped areas, therefore, was excluded as a form of consumption. Re-use of seepage or return waters and water recovered by drainage systems within an area were automatically included and served to decrease the burden on the original river flow. Large irrigated areas were used and records of stream flow were considered of from 5 to 10 years to eliminate minor irregularities of climatic and variations in crops in irrigated lands.

The crop-year draft on soil moisture and the water drawn from ground-water sources, being, respectively, Factors (*b*) and (*d*) of H_v in Equation (7), were not evaluated in Meeker's studies. The values of the valley consumptive use reported herewith, therefore, may be slightly smaller than the actual magnitudes of consumptive use in the respective valleys. Unpublished records supplied the Committee by Meeker contain interesting data concerning gross and net duty of water in 18 river systems, and on 9 of these consumptive-use data are available. The observations on these 9 systems are reported in Table 7.

It will be noted in Column (9) of Table 7 that the consumptive use, defined as dimunition in stream flow, varied from 0.9 acre-ft. per acre per year in the Little Laramie River Basin to 1.3 acre-ft. per acre per year in the Sevier River Basin and on the Boise Project. Column (9*a*) which has been added to Table 7 by the Committee, gives the valley consumptive use as here defined, with the exception noted that there are no measurements of draft on soil moisture or ground-water. Assuming the seasonal draft on soil moisture and ground-water to be 0.3 acre-ft. per acre, as was assumed for the Sevier River Basin, then valley consumptive use varies from 1.5 to 2.2 acre-ft. per acre for the river basins reported in Table 7.

Investigations of the return flow on the Cache la Poudre and South Platte River by Meeker (13) and on the Lower South Platte by Parshall (15) give further evidence concerning consumptive use. In a résumé of factors affecting return flow, Meeker places the consumptive use (not including crop-season rainfall or draft on soil moisture) at 1.25 ft. and reports that from 35 to 65% of waters diverted for irrigation appear as return flow in the Colorado streams mentioned.

Meeker's conclusion, based on rather extensive investigations, is as follows:

> "The conclusion from these studies and from comparisons with the natural return flow from large older irrigated areas is that the amount of water actually consumed by crops is relatively small compared with the amount diverted, and further that with efficient drainage on an irrigated project, the actual plant consumption has decidedly low limits regardless of the water applied."

DISCUSSION

It has been pointed out that many independent factors influence the valley consumptive use. However, there is urgent need for arriving at some relatively simple method of estimating its magnitude. Therefore, in order further to study the merits of Hedke's proposal, the determinations of valley consumptive use made by Crandall, Harding, and Meeker are assembled in Table 8. Each river system is designated by a number given in Column (1). The minimum growing temperatures given in Column (5) were selected on the basis of the minimum growing temperatures for the predominating crops in the respective areas. Column (6) contains the names of the U. S. Weather Bureau stations from which temperature records were taken as the basis for computing the available heat as given in Column (8).

The data presented in Columns (7) and (8) are platted in Fig. 2, which suggests a straight line relation between U_v and Q_h. The slope of the line in Fig. 2, or the magnitude of K in the equation, $U_v = KQ_h$, is 0.00039, which is about 8% less than the magnitude of K as determined from Hedke's analysis of Cache la Poudre experiments. This is considered a close agreement, particularly when it is recognized that the minimum growing temperatures of so many crops under such varied conditions can only be approximated. The Snake River results seem to be the most erratic. It is hoped that this report will elicit additional data on valley consumptive use, particularly from areas of high mean annual temperatures, so that the straight line relation indicated by Fig. 2 may be affirmed or modified.

TABLE 8.—MAGNITUDES OF VALLEY CONSUMPTIVE USE, U_v, AND AVAILABLE HEAT, Q_h, FOR TYPICAL IRRIGATED VALLEYS IN THE WEST.

No. (1)	River system. (2)	Years' work. (3)	State. (4)	Minimum growing temperature, in degrees Fahrenheit. (5)	Cities furnishing temperature records.	Valley consumptive use. (7)	Available heat, Q_h. (8)
1	Little Laramie......	12	Wyoming ..	33	Laramie..................	1.2	3 455
2	Little Laramie......	3	Wyoming ..	33	Laramie..................	1.3	3 465
3	Boise Project, Mason Creek..............	..	Idaho.......	36	Boise, Caldwell, Meridian.	1.4	4 357
4	Cache la Poudre....	2	Colorado ...	..	Fort Collins. Greeley......	1.8	4 150
5	South Platte.........	10	Colorado ...	36	Julesburg, Sterling, Fort Morgan..................	1.8	4 523
6	Boise Project........	3	Idaho.......	36	Boise, Caldwell, Meridian.	1.6	4 357
7	Sevier River.........	8	Utah	36	Richfield, Marysvale	1.5	3 828
8	South Platte.........	2	....	36	(See No. 5)	1.9	4 523
9	Snake River Twin Falls Project......	2	Idaho.......	36	Twin Falls, Shoshone	2.6	4 035
10	Kings..................	4	California..	42	Fresno, Hanford..........	2.0	7 903
11	Kaweah	4	California..	42	Visalia, Tulare, Lindsay..	2.2	7 751
12	Truckee and Reno ..	..	Nevada.....	36	Reno	3.0	5 150

SUMMARY AND CONCLUSION

1.—The term, "consumptive use of water", has been given an important place in engineering literature.

2.—Definitions of consumptive use are proposed in this report both in its basic sense and with respect to the area concerned, whether a farm, a project, or a valley.

3.—Some of the common difficulties in measuring consumptive use are enumerated.

4.—That measurement of project and valley consumptive use under some physical conditions is impractical is indicated.

5.—Investigations concerning consumptive use, as made in California, Colorado, Idaho, Nevada, Nebraska, Oregon, and Utah, and in Alberta, Canada, are briefly reviewed.

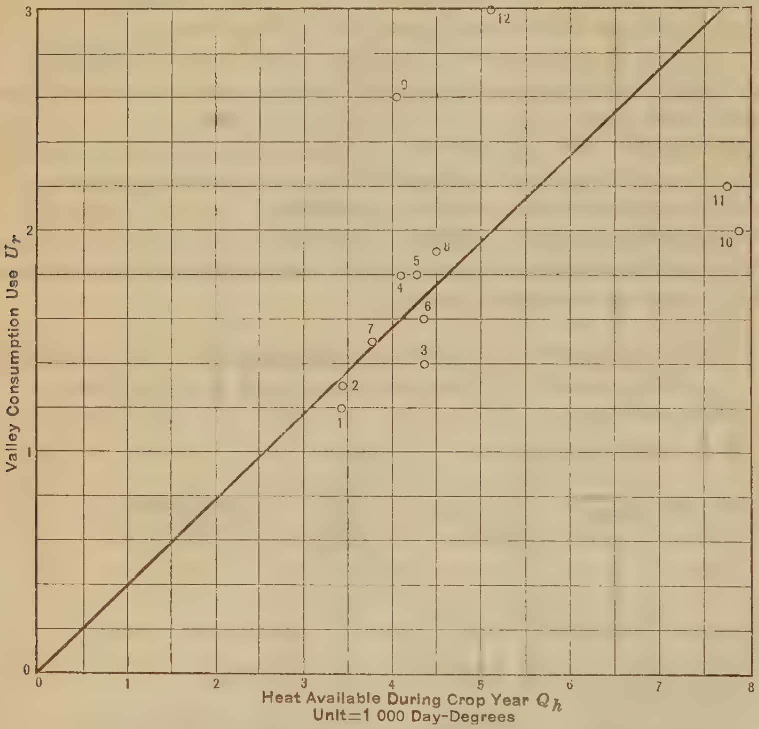

FIG. 2.—VALLEY CONSUMPTIVE USE OF WATER ON TYPICAL PROJECT AS RELATED TO AVAILABLE HEAT.

6.—The engineering reports and publications reviewed contain information which is considered valuable as a guide to approximations of consumptive use. If the factors affecting the consumptive use do not vary greatly, and particularly if the crop yield is approximately the same throughout a valley as on experimental plots within the valley, then the valley consumptive use must be either equal to or greater than the net consumptive use. It, therefore, seems reasonable to believe that quantities of valley consumptive use, U_v, which are less than the net consumptive use, U, if obtained by careful measurements, indicate that the soil conditions and crop yields for the valley are too low, and that, if the average yields for the valley are increased to those secured on experimental plots, the valley consumptive use will also be increased.

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Committee of Irrigation Division
on Duty of Water.

DISCUSSION

HAROLD CONKLING,* M. AM. SOC. C. E. (by letter).—This report on the consumptive use of water in irrigation will be welcomed by the great body of engineers in Western States who are engaged in the problem of water supply for irrigation and domestic use. About 1918 the writer was engaged in a series of studies undertaken by the U. S. Reclamation Bureau for a comprehensive and complete development of some of the major stream systems of the West. The Rio Grande, North Platte, Humboldt, and Colorado Rivers were the principal streams studied.

It was soon apparent that as far as the water supply was concerned, the first effort must be to determine the consumptive use for different areas. The quantity that it was necessary to divert was a detail for any particular project which governed the size of the works, but was not usually a factor in determining the possibilities of a stream system, since consumptive use is not greatly influenced ordinarily by the volume diverted. It also seemed probable that more water is consumed per acre in the warmer climates than in the cooler and some of the streams studied, notably the Rio Grande and the Colorado, furnish water for lands in climates ranging from the high cold valleys of Colorado to the sub-tropical heat at the Mexican border.

A thorough search for data on consumptive use was made because, before the investigations could proceed intelligently, this basic matter of water supply had to be settled. Very few determinations of consumptive use had been made at that time; some of those used by the writer are included in its report, modified by later information. It was believed at the time that those determinations, which were made by measuring the total quantity of water reaching a considerable tract of irrigated land and the total quantity leaving the tract, were the most reliable as they gave directly the average result of all the multitudinous factors that may affect the consumptive use on any individual acre. Accordingly, an investigation of all the Reclamation Bureau projects was made to find such areas as were favorable to determination in this way, but such tracts were very scarce.

Since 1923 the writer has had charge of an investigation of the water supply in San Gabriel Basin of Southern California. The term, San Gabriel Basin, applies to the entire San Gabriel stream system. The streams of this system, like most of those in Southern California, flow across detrital filled valleys of unknown depth, and these valleys form huge reservoirs from which the water supplies of the region are drawn. Without these reservoirs the life of the region as it is to-day would be impossible because long series of dry years are characteristic of the climate. A determination of the consumptive use in the upper of the two detrital filled valleys across which the San Gabriel flows was made by balancing inflow against outflow. In that region—because of the large quantity of water pumped from the underground reservoir—change in underground storage is a much larger and more important item than in the usual determination of this nature in the inter-mountain country. Any attempt to convert the measured change in elevation of water-plane into acre-feet of storage may result in serious error. In this investigation a determination of voids in various classes of materials encountered by the well driller was made and then from well-logs the percentage of voids at each well was calculated. With many well-logs to use, a map showing lines of equal voids was made. When there was superimposed on this map the data showing lines of equal annual change of ground-water level, a calculation of the acre-feet change could be made. Even with this elaborate study, chance for large error exists and unless the water-plane has moved in both directions there is no check to determine such error. In this case the average water-plane fell two years, remained stationary one year, and rose one year. The final results were such that the determination of acre-feet put into or withdrawn from underground storage each year is believed to be reasonably accurate.

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Another factor which affects the apparent consumptive use in any one year is the large difference in rainfall. This affects it in two ways. If the rainfall is deficient the growth of vegetation on unirrigated land is less than normal and, consequently, the actual consumptive use is less. Furthermore, it is probable that, in such a year, more water is released to the water-plane from that part of the vertical section of the valley fill above the water-plane and below the root zone than reaches it from deep percolation of rainfall. Hence, there is a depletion in this area which cannot show in the hydraulic equation and the apparent consumptive use is less than the actual. This possibly is a considerable item as the ground surface over a large part of San Gabriel Valley is 200 ft. or more from the water-plane. It is believed, however, that after two years of nearly the same precipitation, conditions will have reached an approximate state of equilibrium. In Table 9 are shown data as to inflow and outflow for four years (1923-27), the first two of which were very dry and the second two slightly wetter than normal. It is believed therefore that the second and fourth year average should give results that eliminate, to a large extent, the uncertainties noted in the foregoing. By a coincidence the average for four years is about the same as for the two years in question.

One other unknown quantity enters into the determination. The underflow out of the valley is uncertain. The approximate cross-section of the pervious area of the valley outlet is known and the approximate underflow is computed from this knowledge. While the variation from the figure used may be large, it does not seriously vitiate the figure for total consumption. The hydraulic equation is arranged in Table 9 to show the different items entering into it. The resulting "apparent consumptive use" is found by subtracting the total output from the total supply to the valley floor.

It will be noted (Table 9) that the apparent consumption is 191 000 acre-ft. This is believed to be close to the actual consumption, but for various reasons is probably a little larger than the actual (value assumed for underflow treated as definitely known). This supplied the water consumption of 129 600 acres of valley land, of which 81 700 acres are using water (irrigation or domestic), 33 200 acres are irrigable, and 14 700 acres are non-irrigable (country roads and washes). In addition, water is pumped to 3 240 acres of hillside land. Crops are produced as follows: 30 334 acres of citrus trees; 19 758 acres of deciduous trees, mostly English walnuts; 8 831 acres of truck gardens; 2 960 acres of alfalfa; and, 23 111 acres of urban area. Adding the land irrigated on the hillsides to that irrigated in the valley gives 84 940 acres irrigated or using domestic water.

TABLE 9.—CONSUMPTION OF WATER IN SAN GABRIEL VALLEY, IN ACRE-FEET.*

	SEASON.				
	1923-24.	1924-25.	1925-26.	1926-27.	Average.
INFLOW:					
Measured Inflow:					
Mountain run-off	32 470	30 060	141 000	163 000	91 600
Precipitation on valley floor	114 000	128 000	218 000	251 000	178 000
Total measured	146 470	158 060	359 000	414 000	269 600
Unmeasured Inflow:					
Mountain run-off	2 200	2 100	9 700	11 200	6 300
Hill run-off on surface	1 860	1 960	7 260	8 590	4 920
Lateral percolation from hills	0	0	5 750	6 610	3 090
Total unmeasured	4 060	4 060	22 710	26 400	14 310
Total inflow	150 530	162 120	381 710	440 400	283 700
OUTFLOW:					
Measured Outflow:					
Storm water through Narrows	2 380	4 530	52 800	81 900	35 300
Storm water through Arroyo Seco	0	0	240	1 000	310
Pumpage through Narrows	14 800	14 500	12 900	12 200	13 600
Rising water through Narrows	73 200	56 000	49 000	57 200	58 850
Sewer discharge through Narrows	70	4 220	5 130	5 320	3 680
Total measured	90 370	79 250	119 870	157 620	111 790
Unmeasured Outflow:					
Storm water through Monterey Park	200	200	2 000	2 000	1 100
Underflow through Narrows	25 000	25 000	25 000	25 000	25 000
Total unmeasured	25 200	25 200	27 000	27 000	26 100
Total outflow	115 600	104 450	146 900	184 600	137 900
Total inflow	151 000	162 000	382 000	440 000	284 000
Change in underground storage	−113 000	−129 000	+2 400	+58 700	−45 000
Total supply to valley above Narrows	264 000	291 000	380 000	382 000	329 000
Total outflow	116 000	104 000	147 000	185 000	138 000
Apparent consumption	148 000	187 000	233 000	197 000	191 000

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* The annual rainfall in the valley averages 19.56 in., of which 78% falls in the four months from December to March, inclusive.

If the 191 000 acre-ft. consumed is divided by the gross area, the consumptive use is 1.44 acre-ft. per acre. Does this value compare with that given in Column (9*a*) of Table 7 of the report? Nothing is said there as to whether the irrigated areas given in Table 7 are net or gross, but from the writer's knowledge of these various tracts he is inclined to believe that the acreage given is the gross and that the net irrigated is from 75 to 85% of the gross.

To obtain consumptive use in San Gabriel Valley on a comparable acreage basis, it is necessary to estimate what the increased consumption will be when the entire irrigable land is placed under cultivation. The fallow land consumes only rainfall, while the irrigated land consumes both rain and irrigation water. To determine this increased consumption it is necessary to find the differences in the ability of native and cultivated crops to capture rainfall. These differences are more profound than is generally supposed, and work is in progress (1928), which tends to show that in the areas of heavier precipitation the difference in total water consumed by native vegetation and by cultivated crops is not great, except in the case of alfalfa and the area of this crop in the valley is negligible. In the absence of final determination on this point it is here assumed (for the purpose of finding a figure approximately comparable to that used in the report) that the consumption on irrigated areas is 1.9 ft. in depth and that on fallow irrigable land is 0.8 ft. in depth, so that when the valley is entirely irrigated the total consumption will be 36 000 acre-ft. greater than at present, making 227 000 acre-ft. in all.

It is believed that this figure, divided by the gross valley area plus the land irrigated on the hillside will give a value comparable, so far as percentage of area irrigated is concerned, with those used in the report. When this is done it results in a consumption of 1.7 acre-ft. per acre.

While this value is believed to be comparable on the basis of acreage, it may not be comparable in other ways. It is a measure of the total water consumed, whether from irrigation or rainfall, during the entire year, whereas that given in the report is for consumption from rainfall and irrigation during the crop-year. However, if the rainfall had not occurred in San Gabriel Valley, it is believed from other data that consumption of irrigation water (if irrigation had continued through the twelve months) would not have been greatly different from 1.7 acre-ft. per acre. In explanation, it should be stated that the growing season extends throughout the entire year and that cessation of irrigation occurs only because of the winter rains.

Referring to Fig. 2, wherein "heat units" are compared to "valley consumption", it is found that this value for San Gabriel Valley would plat a little lower than that for the Kings River and the Kaweah River areas on approximately the same location as to "heat units". Examination of the data from which this graph was platted shows a substantial disagreement between values from the Truckee and Snake River areas and those for similar climates, such as the Boise, Cache la Poudre, Sevier, and South Platte Valleys. The reason for the excess consumption in the Truckee Valley may rest in the combination of the peculiar underground conditions and the excessive applications, but this does not apply to the Snake River area. An examination of the indirect methods used in securing the data for this area shows that many errors could occur.

The Committee raises the question as to whether a curve of heat units against consumptive use is justified. It may be that it is, if it is restricted to showing the relation between the consumptive use of areas growing essentially the same crops. The crops of the inter-mountain country are mainly alfalfa, grain, and roots, and are essentially the same from north to south, but nothing quite comparable is found in California except in the extreme northern part or in the small areas east of the Sierras. In most places the irrigated crops are different and contain a larger proportion of trees. Then, too, humidity is less in the inter-mountain country than it is west of the Sierras. It would seem improbable that a comparison of consumptive use based on heat units is logical between such divergent types of crops. If the Kings and Kaweah values (see Fig. 2) are neglected and only the values from the inter-mountain region considered, the curve becomes almost vertical. If the Snake River and Truckee values are given little weight—the first because of possible inaccuracies and the second because of unique conditions—there is little basis for a curve. If one is drawn it could as well be in a much different direction than the one which appears as Fig. 2.

The addition of the San Gabriel Valley to the group, its rather close check with the other California areas, and the fair consistency in the type of crops grown in these areas, suggest that while heat units may be a determinant in increasing the consumptive use, the higher type of crop grown in the hotter regions where land values are high requires less water. Therefore, if marketing conditions are such as to justify the higher type of crop, and high land value, a curve of "consumptive use-heat relation" will not be of value when applied to all the different conditions encompassed in the data published by the Committee.

Charles H. Lee,[*] M. Am. Soc. C. E. (by letter).—The subject of this report is an increasingly important one in all regions where agriculture depends on irrigation. It not only enters into questions of the management and economics of irrigation projects, but also water supply problems, both surface and underground. It has become a very important factor in the settling of controversies on great stream systems where the public welfare of valleys, States, and even nations is involved. Not only is correct knowledge of the absolute loss in irrigation of interest to several branches of the Engineering Profession, but also to public officials, legislators, and the public. Therefore, the Committee in selecting the subject of this report has struck upon a very live topic, as well as one which embodies the essence of its work.

The term, "consumptive use of water," although not strictly involving a correct use of the word, "consumptive", has been more or less accepted by those interested in the subject. Like "duty of water" and many other technical terms it may be considered a distinctive identification mark which common usage has sanctioned and the profession has accepted.

The proposed definition of "consumptive use" the writer believes is sufficiently comprehensive, with the possible exception that it does not recognize either double cropping or intercropping. Both are practiced in California, particularly in Southern California and the Imperial Valley, and the annual consumptive loss per acre is materially increased thereby, especially in the case of double cropping. In order to recognize the practice in the definition, the words, "or crops", might be added after the word, "crop".

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[*] Cons. Hydr. Engr., San Francisco, Calif.

The mathematical form for analysis of consumptive use data is somewhat confusing at first reading, but its use is probably justified from the standpoint of completeness and simplicity. The writer has followed this method satisfactorily for many years in working out individual cases, both for determination of consumptive use and for equating ground-water absorption and discharge for valleys of the closed-basin type.

Net Consumptive Use.—The practicability of determining working values for net consumptive use, U, by means of tanks or potometers, is dependent on reproduction of natural conditions. The water supply is under control and is supplied more regularly and uniformly than is possible in practice and in more exact synchronism with plant requirements. This probably tends to increase rather than decrease evaporation and plant transpiration. Also, there is more complete exposure of the plant to light and wind, and there may be the absence of the vapor blanket present under field conditions. The tank in relation to the field is in fact the counterpart of the small land evaporating pan as compared with the lake or reservoir. The work of Sleight, at Denver, Colo.,* and of the writer in Owens Valley, California,† has shown conclusively that the evaporation from a small land pan considerably exceeds that from a lake or reservoir. Similarly, it is to be expected that the transpiration from a crop growing in a small tank not surrounded by growing vegetation of a similar character will exceed that from a cropped field. The conditions surrounding tank experiments should be ascertained before placing dependence on the results.

The method by field plot measurements on the other hand, while it has many advantages over the use of tanks, has the disadvantage that draft on the ground-water and deep percolation loss are both unmeasurable. In the effort to avoid the former, experimenters have endeavored to select sites having great depth to ground-water, while in the case of the latter, small and frequent irrigations have been given in the hope that deep percolation loss would be reduced to zero. In each determination of U presented by the Committee there is more or less uncertainty whether or not deep percolation occurred and whether its value is really so small as to be negligible. Without more convincing proof, it is open to doubt whether in all cases deep percolation can be considered as negligible even with a deep water-table.

Farm Consumptive Use.—The conditions under which farm consumptive use (U_f) occurs, differs but little from that of net consumptive use from field plots. The elements of difference are principally with respect to the application of water in accordance with actual farm practice rather than under experimental control, and frequently a shallower depth to the water-table. As is to be expected, the measured values of U_f are generally greater than those for U determined at field plots. This is due to the inclusion of deep percolation in the former.

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* "Evaporation from the Surface of Water and River Bed Materials," *Journal of Agricultural Review,* Vol. X (1917), pp. 229–230.

† *Transactions,* Am. Soc. C. E., Vol. LXXVIII (1915), pp. 176–181.

Table 10 assembles all comparable values presented in the Committee's report, those of Lewis in Snake River Valley being omitted because of lack of data regarding soil moisture and crop season rainfall. A striking feature is the wide variation in apparent values of U and U_f. This is probably due both to errors of measurement and to difference in crop-growing conditions. Table 10 serves to emphasize the conclusion of Kisselbach that "there is no such thing as a definite water requirement which is constant for any kind of crop", and points clearly to the fact that the acceptance and use of U and U_f data on any basis other than as an aid to judgment are not justified.

TABLE 10.—COMPARISON OF CONSUMPTIVE USE AS DETERMINED FOR FIELD PLOTS (U) AND FOR INDIVIDUAL FARMS (U), IN ACRE-FEET PER ACRE.

Crop.	FIELD PLOTS.				INDIVIDUAL FARMS.		
	Widtsoe, Cache Valley, Utah.*	Snelson, at Brooks, Alberta.†		Widtsoe and Harris, at Utah Agricultural Experiment Station.‡	Hemphill, in Cache la Poudre Valley, Colorado.§		Israelsen and Winsor, in Sevier Valley, Utah.‖
		Minimum.	Maximum.		Average yield.	Maximum yield.	
Sugar-beets	2.5	....	....	2.4	3.0	3.0	3.1
Potatoes	2.2	....	....	2.2	3.3	4.2	2.6
Alfalfa	2.3	1.00	2.62	4.6	3.7	5.8	3.4
Corn	2.5	....	....	2.2	....	....	....
Wheat	2.4	0.85	1.82	2.5	2.1	3.1	....
Oats	2.5	0.72	1.75	3.1	1.4	3.6	....

* Crop season rainfall, 0.42 ft.; draft on soil moisture, 0.1 ft. for corn to 0.83 ft. for alfalfa.

† Crop season rainfall not stated; draft on soil moisture measured, but not stated.

‡ Depth of soil moisture and rainfall, 0.86, 0.52, 1.25, 0.52, 1.15, and 0.81 ft., respectively.

§ Crop season rainfall, 0.92 ft.; draft on soil moisture, 0.30 ft.

‖ Crop season rainfall, 0.30 ft.; draft on soil moisture, 0.30 ft.

The unusually large values for U_f found by Hemphill in Cache la Poudre Valley, where the crop season rainfall was 0.92 ft., or more than twice that in any other location, raise a question as to the propriety of adding the full depth of the observed rainfall. The writer's own observations and a study of the observations of others lead to the conclusion that most or all of the summer precipitation in valley parts of semi-arid and arid regions is lost, either through surface run-off or immediate evaporation from foliage and the soil; and that very little percolates to a depth sufficient to make it available to any but the most shallow rooted crops. In view of this condition the addition of the total rainfall for the crop season does not carry out the purpose of the Committee, which was to arrive at basic values capable of use in any locality. It is believed that this item needs experimental study and that, pending the availability of further data, consumptive use values which include total rainfall should be used with caution. This suggestion obviously applies to consumptive use in all its phases—basic, farm, project, and valley.

Project and Valley Consumptive Use.—The determinations of "project use" (U_p) and "valley consumptive use" (U_v), as defined by the Committee, are far from simple problems. Unlike U_f, in which the ultimate disposition of D_f is immaterial, in the case of U_p and U_v it is necessary to trace D_p and D_v at least to the extent of ascertaining the quantity returning to stream channels and the evaporation and transpiration loss from non-cropped land. Both U_p and U_v should be less than U_f, since a portion of D_f is accounted for where larger areas are considered. In the case of the project the unaccounted portion is that continuing as percolating water beyond the project boundary to appear later in a surface stream or pass into the atmosphere from an area of shallow ground-water. For valley consumptive use the unaccounted portion, if any, passes from the valley as sub-surface flow. If there is no sub-surface flow, as in the case of a closed geologic basin, U_v should be equal to U. The complications surrounding the determination of the values of U_p and U_v arise, first, from the inability to see the geologic formations beneath the surface of the ground; second, from the hidden course and slow movement of deep percolating water from irrigated land; and, third, from the mingling of such water with percolating water from other sources. An analysis of the Committee's report shows that the proposed method of determining U_p and U_v-values is still in a formative state and that the complications which are encountered in practical application are being dealt with either at arm's length or not at all.

The earlier investigators attacked the problem in its simplest form and attained roughly approximate results. Later investigators, recognizing the omissions and assumptions, have introduced refinements. The Committee's report is the latest step in this process and represents a notable advance. It is to be recognized, however, that practically all investigators have thus far approached the subject from the viewpoint of irrigated agriculture. From this background, the basic units of time and area have been the crop year and the cropped area. These units are essentially artificial and until natural units are adopted the concepts must continue to be in terms of special cases.

The next step forward is the adoption of the climatic year, instead of the crop year; and the topographic or geologic unit of area, instead of the project or project group area. The following examples will serve to illustrate the advantages of these units. In most localities where irrigation is practiced the crop year covers six to eight months, or from one-half to two-thirds of the full year. Due to the slow movement of ground-water there is a considerable lag between the diversion of water from a stream and its return as seepage or spring flow. Water which passes beyond the plant roots joins the general ground-water body below (thereby raising the water-table), and remains in sub-surface storage until the water-table returns to its initial level. The lag period may vary from a few weeks to five or six months, depending on the character of the subsoil, the relative elevation of irrigated land and adjacent drainage channels, and the distance of the irrigated land from drainage channels flowing direct to the main stream. Observations which do not extend over the lag period fail to indicate the amount of delayed return reaching the outlet stream or evaporated from moist non-cropped soil during the non-irrigation period, and hence give consumptive use values which are too great. The writer's observations indicate that in certain cases the degree of error from this source may be very large and that it is seldom negligible.

Another illustration in which the scope of the Committee's analysis might be broadened is with respect to area. Consideration of inflow and outflow has been limited to the cropped area and the immediately adjacent land from which evaporation or transpiration from natural vegetation occurs. The attempt has been made to list the *H* and *R*-factors with respect to a single project area or group of project areas within a valley. This procedure has given rise to difficulties of measurement. For instance, it is impossible in practice to measure the sub-surface flow into the crop area. This is recognized by the Committee in commenting on the many valleys of the Southwest where inflowing ground-water is important. The statement is made that, in such instances, it is "impractical to obtain dependable measurements of project or valley water supply."

Although not mentioned, the same difficulty exists with respect to subsurface flow from the cropped area. Some of this may appear as seepage or spring flow and be measured as outflow, but, on the other hand, there may be loss by evaporation from shallow water-table outside the project area, or subsurface outlets other than the adjacent stream. There may also be storage accumulation as represented by rise of the water-table in non-cropped areas outside the project. By approaching the problem from the aspect of a topographic or geologic unit, however, these difficulties largely disappear.

The illustrations are sufficient to indicate the errors into which the investigator may fall if the methods outlined by the Committee are his only guide. It is not the writer's purpose to outline a method to replace that of the Committee, for the latter is well adapted for use in many cases, but rather to suggest a broader foundation for analyzing the individual problem more comprehensively and if necessary to modify the Committee's method to fit the case.

Since about 1910 there has been a notable accumulation of knowledge in the field of ground-water hydrology. The existence has been shown of many natural ground-water reservoirs occupying geologic basins more or less coextensive with topographic valleys.* The source, movement, and ultimate disposition of the ground-water in these basins have been traced and methods have been developed for quantitative measurement of inflow and outflow.† These methods have now become sufficiently standardized for practical use in determining the dependable yield from ground-water sources.

In 1926 the City of Los Angeles, Calif., completed a program of ground-water development in the Independence Region of Owens Valley the extent of which was originally planned from the results of a quantitative ground-water study made under the direction of the writer from 1908 to 1911. The development work, consisting of wells and pumping plants, cost approximately $250 000 and yields an auxiliary supply to the Los Angeles Aqueduct System of more than 100 sec-ft. The same investigational methods have also been applied to the solution of other problems. It is the writer's conviction that the methods of ground-water hydrology used in making quantitative determination of ground-water supply have direct application to the problems of project and valley consumptive use. They will serve to point out unsuspected items in the inflow-outflow equation, to evaluate unknown factors, and to make possible a solution in valleys with inflowing ground-water which is claimed to be impractical by the Committee.*

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* "Occurrence of Ground-Water in the United States," by O. E. Meinzer, *Water Supply Paper 489*, U. S. Geological Survey, pp. 291–303, 1923.

† "Water Resources of Part of Owens Valley, California," by Charles H. Lee, *Water Supply Paper 294*, U. S. Geological Survey, 1912.

In adapting the methods of ground-water hydrology to the measurement of consumptive use, the first step is a geological survey to determine the character and extent of the impervious bed-rock basin, the limits of the hydrologic water-shed, the character and distribution of the porous alluvial fill, and the probability of sub-surface inflow or outflow. A more or less complete water-table map with typical fluctuation graphs covering the annual cycle should also be available, and a general knowledge of local precipitation, run-off, stream channels, irrigated land, seeped or swamp land, springs, etc. With such information it should be possible to list all ground-water sources and outlets and to ascertain whether there are any negligible or compensating items so that the problem can be reduced to its simplest terms.

Three general types of valleys are met in consumptive-use determinations:

1.—There are those without underlying alluvial fill in which the bed-rock formation is covered with a thin veneer of decomposed rock and soil in place. This type seldom occurs in practice. If the bed-rock is non-porous rock, such as granite, the valley can be treated under one of the other two classifications. For a porous, fissured, or cavernous bed-rock, the measurement of consumptive use may be impracticable.

2.—There are valleys of the "open-basin" type in which the underlying bed-rock basin is filled with porous alluvial material having one or more areas of direct contact with outside porous formations which may extend even beyond the topographic drainage area. This type also includes valleys where the bed-rock is leaky, due to the occurrence of porous or soluable bed-rock, such as open sandstones, limestones, etc.

3.—There are valleys of the "closed-basin" type, in which the porous alluvial fill has no connection with outside formations, being completely enclosed by non-porous bed-rock bottom and sides continuous with the surrounding hills or mountains.

Proposed Method of Determining U_v.—To illustrate the application of the proposed method of determining valley consumptive use, the various quantities will be defined and equations formulated. The period of time for which equations are set up is the climatic year and may be October 1 to September 30, November 1 to October 31, or January 1 to December 31, depending on local precipitation and run-off conditions.

The area, in the case of a closed geologic basin, is the superficial area of the porous alluvial fill, usually extending to the margin of the valley at the base of the surrounding hills or mountains. For an open geologic basin, the porous material of which extends beyond the margin of the topographic valley, the limits of the area may be chosen with respect to practicability of measuring or estimating the sub-surface inflow and outflow, although they should be restricted as much as possible.

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* "Quantitative Methods of Estimating Ground-Water Supplies," by O. E. Meinzer, *Bulletin*, Geological Soc. Survey, 1912.

The list of quantities has been designated by symbols which correspond wherever possible to those used by the Committee, as follows:

For annual inflow:

I_v = measurable surface stream flow, in acre-feet per cropped acre, entering the valley at the margin, including both main and lateral streams.

F_v = measurable or estimated sub-surface flow, in acre-feet per cropped acre, entering the valley at the margin.

P_v = the portion of direct precipitation falling upon the surface of the porous valley fill, which is not immediately evaporated and contributes to direct surface run-off, soil moisture, or ground-water, expressed in acre-feet per cropped acre.

For annual outflow:

R_v = measurable surface stream flow, in acre-feet per cropped acre, at valley outlet.

D_v = measurable or estimated sub-surface flow, in acre-feet per cropped acre, escaping from the valley, usually at the lower margin, although possible at other points.

U_v = evaporation and transpiration from cropped valley land, in acre-feet per cropped acre.

E_v = evaporation and transpiration loss from non-cropped valley land and natural vegetation, in acre-feet per cropped acre.

For annual sub-surface storage:

G_v = addition to or withdrawal from valley ground-water storage, in acre-feet per cropped acre.

S_v = addition to or withdrawal from valley soil water, in acre-feet per cropped acre.

The relation upon which equations are based is the annual equality for any valley of inflow and outflow with appropriate correction for sub-surface storage in the porous valley fill.

Expressing this equation by symbols:

$$I_v + F_v + P_v = R_v + D_v + U_v + E_v \, (G_v + S_v)$$

Solving for U_v, there results the general equation:

$$U_v = I_v + F_v + P_v - R_v - D_v - E_v \, (G_v + S_v)$$

For valleys of the closed-basin type, this reduces to the form:

$$U_v = I_v + P_v - R_v - E_v \, (G_v + S_v)$$

In arid regions where direct precipitation does not contribute appreciably to surface run-off, ground-water, or soil moisture, the equation reduces still further to the form:

$$U_v = (I_v - R_v - E_v) \, G_v$$

If the area of cropped land is of minor importance and it is desired to determine losses from natural vegetation, the last equation may take the form:

$$E_v = I_v - R_v - U_v \, G_v$$

To evaluate E_v in terms of acre-feet per acre of natural vegetation and seeped land, it should be multiplied by the ratio of the area of cropped land to that of natural vegetation.

The I_v and R_v-quantities are capable of measurement by ordinary stream-flow methods. The F_v and D_v-quantities are difficult and expensive to measure, but as they are ordinarily small in amount, they can often be estimated from data available from geological and water-table surveys. For closed basins they are zero.

The value of P_v although difficult to determine in semi-arid and humid regions is capable of measurement. Direct addition to run-off from precipitation on the valley floor can be determined from continuously recording water-level gauges on streams. Immediate addition to ground-water can similarly be determined from shallow well records and porosity data. Soil moisture additions are easily determined by ordinary soil moisture tests made before and after storms. The extent to which it is advisable to go in making detailed observations to determine the distribution of direct precipitation upon the valley floor is a question to be answered in each individual case.

The value of E_v is usually quite small relative to U_v and can be determined with sufficient accuracy from a knowledge of the area of moist land and natural vegetation, from local water evaporation, and from the results of observations in other localities.*

For determination of ground-water storage quantities, G_v and S_v, the standard methods can be used. Ground-water storage can be determined either from water-table fluctuations, as measured in shallow wells and test holes and porosity data, or by indirect methods, such as those used by Harding and referred to in the Committee's report. It may also be eliminated from consideration where observations of other quantities covering several years are available and ground-water conditions as modified by irrigation have become stabilized. The variation in soil moisture from year to year is in most cases so small that it is negligible, and S_v may be eliminated altogether.

Many special cases are encountered in practice, which permit of modifications of the general equation, in addition to the simplifications stated. Harding, in his San Joaquin Valley work, follows essentially the method outlined by the writer; he adopts an interesting modification with respect to the ground-water storage correction. In his work in San Luis Rey and in other valleys in San Diego County, California, the writer used still other modifications, as suggested by the local synchonism in the periods of run-off and ground-water fluctuation.† Every locality and valley has its special conditions and in almost every case study will disclose important simplification in the application for the general formula.

Committee's Discussion and Conclusion.—The proposal to study consumptive use in conjunction with heat available during the crop year is logical. Temperature is by far the most important factor governing evaporation, the direct straight-line relation between the two having been clearly shown by

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* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), pp. 334–338.

† "Geology and Ground-Water of the Western Part of San Diego County, California," by A. J. Ellis and Charles H. Lee, 1919, pp. 142–155.

Sleight* and other investigators. Transpiration being a special form of evaporation is controlled largely by temperature as is also the plant life process of which transpiration is a phase. The straight-line relation between valley consumptive use and heat available during the crop year, which is indicated by Fig. 2, is therefore a reasonable assumption. It should be pointed out, however (in view of the present uncertainties regarding consumptive use values as determined without the aid of a standard method by various investigators, as well as the limited range of available heat values to draw from), that at present neither the straight-line relation nor the equation of any line can be considered as established. The first requisite is dependable data; the second, sufficient data.

The writer's experience leads him to modify the Committee's Conclusion 4 "that measurement of project and valley consumptive use under some physical conditions is impractical", to read, "that measurement of project and valley consumptive use is possible under most physical conditions met in practice." Under the method proposed by the writer, the determination of valley consumptive use in the only case specifically excluded by the Committee as impractical, is made entirely feasible.

The last conclusion (6) of the Committee would appear to be correct provided the data are accurate. The writer, however, does not believe that the data now available are sufficiently dependable to permit conclusions of such far-reaching importance to be drawn.

Results of Measurements in Owens Valley, California.—The Owens Valley, California, is a desert valley typical of the Great Basin. It lies at the eastern base of the Sierra Nevada Mountains with an elevation of about 4 000 ft. The run-off from melting snows on these mountains has provided water for the irrigation of approximately 50 000 acres of land in the valley as well as the major part of the supply of the City of Los Angeles. This valley, because of its physical and meteorological conditions and the wealth of hydraulic data available, offers opportunities for determining values of U_v and E_v. The writer has had intimate familiarity with it since about 1905 and presents the following from personal knowledge.

The valley is a closed geological basin from which the only natural escape for water in appreciable quantities is by evaporation and transpiration. Precipitation on the valley floor varies from 3 to 5 in. annually, most of which occurs during the winter months. The average annual depth of evaporation from a free water surface is 67 in. Many lateral streams fed by snows falling upon the eastern slope of the High Sierra enter the valley along its western border. Such waters as are not locally consumed in irrigation find their way into the Owens River which flows down the trough of the valley and finally empties into Owens Lake. The latter is without surface or sub-surface outlet and is a great evaporating pan from which all residual waters of the valley are dissipated into the atmosphere. Its fluctuations are an accurate index of meteorological and cultural changes in the valley above.

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* "Evaporation from the Surfaces of Water and River Bed Materials," *Journal of Agricultural Research,* Vol. X, 1917, pp. 229–230.

Owens Valley is divided into three nearly equal parts by two ranges of granite hills lying across the axis of the valley and extending from the Sierra Nevada Mountains almost to the Inyo Range on the east. These ranges are known as the Poverty Hills and the Alabama Hills. The upper part of the valley thus subdivided has been termed the Bishop-Big Pine Region, the middle part, the Independence Region, and the lowest subdivision, the Owens Lake Region. The alluvial fill of the valley in the narrow necks opposite the east ends of these ranges of hills is composed of fine material consisting of fine sand, silt, and clay. The hydraulic grade of the main valley is so flat that subsurface flow from one region to another is negligible. Each of these smaller valleys, therefore, constitutes a geologically closed basin from which the only escape for water is the surface flow of Owens River, or by evaporation.

When white men first settled in the valley, the higher lands were dry and sandy, supporting a sparse growth of desert shrubs. Stream borders and the broad bottoms of the Owens River supported meadow grass and willows. Green meadows also occurred along the valley borders in the vicinity of springs. Depressions in the valley floor supported coarse grasses, and occasional moist alkali areas occurred where little vegetation grew. Irrigation commenced in the early Sixties and by 1875 the available supply from the lateral streams was being led out in small ditches on to the adjacent lands for the raising of grain and general field crops. In the late Seventies the construction of large ditches to divert the waters of Owens River was commenced. The irrigated area was rapidly extended, alfalfa being introduced in the Eighties. By 1905, the available flow had been completely diverted and since then no further important extensions have occurred.

The effect of irrigation has been to raise the water-table in the vicinity of irrigated land and to greatly extend the meadow-grass and alkali areas of the main valley floor. The area of Owens Lake, on the other hand, has decreased and its level has been permanently lowered. As an incident to the latter, the Owens River has cut back on a new grade, developing a new channel in the river bottom, varying in depth from 20 ft. near the margin of the lake to zero at a point northeast of Bishop, approximately 70 miles up stream. The new conditions brought about by irrigation, including the water-table and return water, had stabilized by 1905. The condition thus described existed prior to the diversion of water from Owens River through the Los Angeles Aqueduct, which commenced in 1913. The latter did not become important in its effect on Owens Lake until 1918.

U and E_v in the Independence Region.[*]—The Independence Region, during the period of intensive water supply study from 1908 to 1913 prior to the completion of the Los Angeles Aqueduct, had a total area (from which soil evaporation and transpiration occurred) of 37 950 acres. This acreage was made up of 3 010 acres of irrigated land planted to alfalfa and general crops, and 34 940 acres of natural meadow, salt-grass, and moist alkali land. The irrigated area consisted of ten or more detached groups of ranches on as many small streams, making it impracticable to determine a value for U_v. The conditions were ideal, however, for determining E_v, a quantity which is usually indeterminate.

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[*] *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 148.

The water-table beneath most of the irrigated land was too far below the surface to furnish an appreciable supply to crops, and the Owens River did not receive measurable sub-surface accretions. The water absorbed by the porous alluvial fill found its escape by evaporation and transpiration from meadow, salt-grass, and alkali land moistened by capillary water drawn up from the water-table. There was also escape in the form of spring flow reaching the main drainage channel of Owens River. The average annual percolation into the porous material of the valley which reached the water-table, as determined by the writer from observations made during a three-year period, was as follows:

Source.	Acre-Feet.
Direct precipitation on mountain slopes between canyons...	19 700
Direct precipitation on outwash slopes....................	9 780
Seepage loss from lateral streams.........................	57 100
Return from irrigation....................................	13 000
Absorption from flood water...............................	6 500
Total	106 080
Deduct spring discharge...................................	22 400
Net consumed by meadow, grass, and moist soil.............	83 680

$$E_v = \frac{83\ 680 \text{ acre-ft.}}{34\ 940 \text{ acres}} = 2.40 \text{ ft.}$$

A check determination based on a detailed survey of depth to water-table and two years of tank observations of evaporation and transpiration from typical sods with differing depth to the water-table, resulted as shown in Table 11.

TABLE 11.

Depth to water-table, in feet.	Area, in acres.	Average depth to ground-water, in feet.	TRANSPIRATION-EVAPORATION.	
			Feet.	Acre-feet.
3	7 610	2.5	3.48	26 400
3 to 4	11 300	3.5	2.80	31 600
4 to 8	16 030	5.5	1.32	21 100
Total...............	34 940	....	2.26	79 100

It is believed that the second value of 2.26 ft. for E_v is closer to the truth than the first, as the detailed observational control was more complete. In the first set-up the only item based on systematic and detailed observation was seepage loss from lateral streams constituting 54% of the total. The other items were in part based on estimates. All items involved in the second set-up were the result of observation. Unusual precautions were taken to have natural conditions for the tank experiments.

U for Alfalfa in Independence Region.—Observations were also made near Independence, on August 9, 1910, to determine transpiration losses from alfalfa in a typical field stand. The method was that used by certain European investigators—determining the rate of loss of water from freshly cut plants, on the assumption that this would approximately represent the rate of loss immediately prior to cutting. Four samples were cut from measured areas at different times during the day. These were weighed at short intervals until no further appreciable loss occurred. Visible wilting did not occur during the first 15 min. after cutting, and this period was used in making calculations. The results were reduced to a 24-hour average, using a factor based on standard transpiration experiments. The local growing season extends from April 15 to September 30, and the production of dry hay amounted to about 5 tons per acre with three crops. The samples used for observation were almost ready for the second cutting. Assuming that the average transpiring surface during the entire growing season was 50% of that on the day of the experiment, the total loss for the season would have amounted to 935 lb. of water per lb. of dry hay, or 3.4 ft. in depth. The total depth of water applied in irrigation on this field during the crop season was 16 ft.

This value for U is consistent with the quantity of water available and the results of tank experiments elsewhere. The method, however, involves unnatural conditions. The samples were dried at the margin of the field out of the main vapor blanket, and the ends of the stems were cut, thus interrupting the life processes of the plant. It is not believed that the results by this method are any more dependable than those obtained by the use of standard tanks.

U_v and U_p in the Bishop-Big Pine Region.—The moist area of the Bishop-Big Pine region, although including considerable meadow and salt-grass, is primarily irrigated land, and the physical conditions are favorable for determining values for U_v. The region as here considered extends from Pleasant Valley on the north to the Poverty Hills on the south, including both sides of the valley. Stream measurements are sufficiently complete for the six years, 1918 to 1923, to determine total inflow from various streams at the margin of the valley as well as outflow in Owens River at Charlie's Butte. Acreage data were also quite complete. The water-table was shallow and D_v negligible. The soils vary from those of a coarse sandy granitic character to black loam. Water was taken from Owens River by means of thirteen ditches, the annual net head-gate diversion varying in depth from 9 to 4 ft. on the land, depending on the position of the ditch on the stream and the available supply. The average annual duty for the larger ditches with earlier rights exceeded 6 ft. The length of the irrigation season averaged 213 days, from March 15 to October 15. The land was irrigated from the Owens River, as follows:

Crop.	Acres.
Alfalfa	8 174
Orchard	850
Annual crops	4 684
Pasture (not cultivated)	20 809
Total	34 517

Approximately, 6 340 acres had a supplemental supply from Bishop and Big Pine Creeks, and, in addition, 10 427 acres, mostly in alfalfa and general crops, were irrigated entirely from creek ditches.

The total area of irrigated land, moist soil, and water surface during the period, 1918 to 1923, based on surveys by the City of Los Angeles and the U. S. Reclamation Service is:

Land Under Canals and Ditches:

Irrigated (cultivated and pasture)	42 984	acres
Corrals, roads, etc.	1 590	"
Tule and willows	1 960	"
Water surface (canals)	450	"
Salt-grass and rabbit-bush	5 405	"
Total	52 389	acres

Land Not Under Canals:

Meadow and salt-grass	2 465 acres		
Water surface (river, creeks, and lakes)	930 "		
Total		3 395	acres
Grand total		55 784	acres

The daily inflow into the valley from Owens River and Bishop-Big Pine and Baker Creeks, and several large springs, is summarized in Table 12, together with the outflow at Charlie's Butte. The inflow figures also include estimates for Horton, Birch No. 1, McGee, Tinemaha, and Birch No. 2 Creeks, aggregating 10% of the total inflow. The period of observation, although including several dry years, was characterized by wasteful use, and it is believed that the results for net loss are representative of average conditions for this valley. The period of observation preceded any appreciable withdrawal of land from irrigation by the City of Los Angeles.

The total evaporation and transpiration loss of 168 600 acre-ft. (Table 12), if spread over an area of 55 784 acres, indicates a value for U_v of 3.0 acre-ft. per acre.

In explanation of the apparently large value of U_v, the following classification of moist lands in the Bishop-Big Pine Region, made by the U. S. Reclamation Service in 1920, is illuminating:

Seepage water on surface	5 450	acres
Water, 0 to 2 ft. below surface	24 480	"
Water, 2 to 4 ft. " "	11 250	"
Water, 4 to 6 ft. " "	13 720	"
Total	54 900	acres

This statement shows an unusually large percentage of badly seeped land from which a high rate of evaporation and transpiration loss occurs. Much of this seeped land is classed in the area already noted as "irrigated pasture". The effect on U_v of the wasteful methods of irrigation and the paralleling of earthen ditch systems at successive levels, one above the other, is apparent. The value tends to confirm the results obtained by Harding in Reno Valley where methods of irrigation and meteorological conditions are similar. In both cases, U_v could be reduced materially by the reconstruction of the ditch systems and improved methods of irrigation.

TABLE 12.—INFLOW AND OUTFLOW FOR BISHOP-BIG PINE REGION OF OWENS VALLEY, IN ACRE-FEET.

Year.	Total inflow.	Outflow.	Difference.
1918	400 200	260 800	139 400
1919	355 500	197 600	157 900
1920	348 400	170 600	177 800
1921	345 300	166 900	178 400
1922	481 900	294 200	187 700
1923	341 500	171 100	170 400
Average	....	....	168 600

The change in area and specific gravity of Owens Lake between 1878 and 1909, during which period the thirteen ditches diverting water from Owens River were built and put into operation, affords a method for determining the value of U_p for the area in the Bishop-Big Pine Region covered by these ditches. The shrinkage of the lake surface and increase in specific gravity of the water with the corresponding reduction of evaporation, should represent the combined loss from the newly irrigated land. An element which might materially modify the result, however, is the extent to which the channeling of Owens River on the new grade established by the lower lake level has reduced the evapo-transpiration from the river bottoms. Prior to irrigation in the valley the Owens River spread out more or less over its bottoms in high water for a width of ¼ to ½ mile, and at all times had a shallow, meandering and frequently subdivided channel. The bottoms were thickly overgrown with willows and grass from which large losses occurred. This condition was greatly changed for a distance of more than 70 miles by the cutting of the new channel. The water level in the river and the adjacent water-table are both lower than formerly, and flooding occurs only in very wet years. Also, the river channel is narrower and more concentrated. This results in great decrease in the amount of vegetation and change in its character, with reduction of losses. The degree to which these changed conditions confuse the results for irrigation is not known, but the data will be presented for what they are worth, since the method is unique and may be applicable in other localities.

The changes which took place at Owens Lake have been carefully determined by the writer and are as shown in Table 13.

The lake being without outlet, is highly mineralized, and its specific gravity has exceeded that of sea water even at the highest historic lake level. A long record of lake levels is available, together with accurate area and capacity curves of the lake basin, a "specific gravity-lake level" curve and a "specific gravity-evaporation" curve.*

TABLE 13.—CAPACITY CHANGES IN OWENS LAKE.

Period.	Stabilized lake level, in feet.	Area of lake surface, in acres.	Specific gravity of lake water.	Annual depth of evaporation, in inches.	Total evaporation, in acre-feet.
1872-78........	3 597	71 142	1.04	64.6	384 000
1909-15........	3 576.5	61 600	1.098	61.1	313 500
Difference....	20.5	9 542	.....	....	70 500

The value of 70 500 acre-ft. for the reduction in annual evaporation from Owens Lake between 1878 and 1909 the writer regards as reasonably accurate. If all the reduction was occasioned by the increased area of irrigated and seeped land, the value of U_p for the net area irrigated from river ditches of approximately 34 517 acres, is 2.00 ft. As this value is considerably less than U_p as determined previously, it is possible that the shrinkage in flooded river bottom-lands along 70 miles of river channel, due to channel cutting, should also be taken into account in order to get the true value of U_p. The detailed conditions along the river as they existed prior to 1878 are so difficult to determine, however, that it is doubtful whether it will ever be possible to get an accurate value for the correction.

Results of Measurements in San Luis Rey Valley.—The San Luis Rey Valley is a coastal valley on the Southern California coast about 50 miles north of San Diego. It extends from the seashore into the mountainous interior for a distance of more than 20 miles, varying in width between steeply sloping hills from 300 ft. to nearly 2 miles. The grade of the valley floor averages 20 ft. per mile, being least near the coast. The run-off from precipitation, mostly in the form of winter rainfall on a drainage area of 565 sq. miles, discharges into the valley from various tributaries and flows thence to the Pacific Ocean. The flow of the San Luis Rey River is confined to storm periods during the winter months, except at the head of the valley where there is a small living stream.

The bed-rock formations underlying and surrounding the valley are relatively impervious and no leakage into them occurs from the porous alluvial fill immediately underlying the valley floor. The fill of the main valley has a superficial area of 7 646 acres and its maximum depth varies from 100 to 200 ft., thinning out to zero at the margins of the valley. The material is largely water-bearing sand with gravel at lower levels. The water-table stands near the surface, the depth varying at different points from zero to 10 or 12 ft. The average annual fluctuation of the water-table is 3 ft., the source of supply being percolation from the flow of San Luis Rey River and direct rainfall. The escape is by evaporation from moist sand and soil, and by transpiration from meadow and salt-grass and from willow, alder, cottonwood, and sycamore trees. Underflow, as measured, was found to be so slow that sub-surface escape to the ocean is negligible. The total area from which evaporation and transpiration take place is 6 640 acres.

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* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 333.

The annual precipitation on the valley floor varies from 10 in. at the coast to 12 to 16 in. at the head of the valley above Pala. Most of this falls during the winter months. The annual depth of evaporation from large bodies of water varies from 58 in. near the coast to 66 in. farther inland.

The writer made an intensive study of the underground water supply of the San Luis Rey Valley during the years, 1913 to 1916, and from these data it is possible to determine E_v for the valley. During the period of observation the area of cultivated crops did not exceed 4% of the total evapo-transpiration area, so that separate classification for irrigated land is unnecessary.

TABLE 14.

Item.	Location.	Character of vegetation.	Method of measurement	Depth, in feet.
U..........	Independence Region of Owens Valley.	Alfalfa.	Weighing freshly cut plants.	3.40
E_v........	Independence Region of Owens Valley.	Natural meadow, salt-grass, and moist alkali soil.	Large-scale measurements of percolation.	2.40*
E_v........	Independence Region of Owens Valley.	Natural meadow, salt-grass, and moist alkali soil.	Soil tanks and water-table survey.	2.26*
U_v........	Bishop-Big Pine Region of Owens Valley.	Irrigated crops, pasture and seeped land.	Large-scale measurements of H_v, R_v, and E_v.	3.00†
U_p........	Bishop-Big Pine Region of Owens Valley.	Irrigated crops and pasture.	Shrinkage in area and increase of specific gravity of Owens Lake.	2.00‡
E_v........	San Luis Rey Valley.	Natural meadow, and salt-grass land, willows, etc.	Large-scale measurement of H_v and R_v.	2.19*

* Based on total acreage of natural vegetation plus very small cropped acreage.

† Based on total cropped acreage plus irrigated pasture, seeped land, and water surface.

‡ Based on total cropped acreage plus irrigated pasture. No correction for reduced river-bottom area.

There being no leakage and negligible underflow to the ocean, the valley may be considered a closed basin. The annual fluctuation of the water-table is within definite limits and storage fluctuation due to wet and dry years is small. With these conditions, E_v may be considered as equal to the average annual volume of water reaching the water-table by percolation from run-off and direct precipitation. Without going into details which are fully set forth elsewhere,* it was found that the average annual recharge based on three years' records, was 14 450 acre-ft. This recharge, spread over 6 640 acres of moist land, gives a value of E_v of 2.19 ft. A check determination made independently and based on a detailed classification of moist areas and the application of the principles controlling soil evaporation and transpiration, as developed in the Owens Valley work and elsewhere, but using local rates for water evaporation, indicated an annual loss of 13 980 acre-ft., or a value for E_v of 2.10 ft.

Table 14 summarizes the values for U, U_p, U_v, and E_v, as found for the Owens and San Luis Rey Valleys.

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* *Water Supply Paper 446*, U. S. Geological Survey, pp. 149–153 (1919).

M. R. Lewis,* M. Am. Soc. C. E. (by letter).—The consumptive use of water is a very definite element of great importance in irrigation, which has long been recognized in regions such as Southern California, where all the available water has been pumped from the pervious fill of closed valleys. More recently its importance has been recognized in studies of the water supply available for whole valleys rather than for individual projects.

The very great increase now taking place (1929) in pumping from wells for drainage and to supply supplementary water is certain to make the determination of consumptive use of vital importance in many areas. Heretofore, the water supply for an irrigation project has generally been based on the quantity of water required to be delivered to the farmers' head-gates plus transmission losses. If, in the course of time, return flow to the stream by natural seepage or by gravity drains has warranted, and if lower lands were available, the irrigated area has been extended. In places where pumps are used for drainage the water becomes available for use on the lands which are being drained or on other lands at approximately the same elevation. In every such case the question arises as to whether there is an excess of either land or water. Old data on the duty of water do not apply. For these reasons, the report of the Committee is especially opportune.

The writer wishes to suggest a few additions to and modifications of the detailed statements of the report.

The terms, D_f, D_p, and D_v, all include seepage losses from ditches and canals in the respective areas. A specific statement to this effect would have added to the clarity of the report.

If in determining H_p any negative value of (c), the depletion of groundwater, is considered, it should not also be considered in the determination of R_p in (b), such deep-soil percolation losses as contribute directly to a measurable rise in the height of the water-table.

The writer believes that it would be better to define U_p as being equal to $\frac{W_p}{A_p}$ and V_p as equal to $\frac{W_v}{A_v}$. Then, Equations (4) and (6) become,

$$U_p = \frac{A_{cp}}{A_p} U + D_p + E_p \qquad (8)$$

and,

$$U_v = \frac{A_{cv}}{A_v} U + D_v + E_1 \qquad (9)$$

in which, A_{cp} and A_{cv} are the cropped areas in the project and valley, respectively; A_p and A_v are the total areas in the project and valley, respectively; and W_p and W_v are the volumes of water, in acre-feet, consumed in the project and valley, respectively.

If these equations are used, the error in defining the word "consumptive", as discussed in the report, would not occur and Equations (5) and (7) would be numerically correct.

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* Irrig. and Drainage Specialist, Oregon State Coll., and Div. of Agri. Eng., U. S. Dept. of Agriculture, Corvallis, Ore.

The writer is glad to see so many different studies of consumptive use brought together in a single report. The conditions leading to inaccuracy inherent in plot determinations of evapo-transpiration are realized. It is believed, however, that the difficulty of preventing or even of recognizing deep percolation losses in plot experiments is so great that it overshadows the errors in large-scale pot experiments on the one side and in valley consumptive-use studies on the other.

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Paper No. 1761

INTERSTATE WATER MATTERS

PROGRESS REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION*

With Discussion by Messrs. R. I. Meeker and M. C. Hinderlider

The Chairman of the Committee on Interstate Water Matters submits the following progress report.

At the Summer Meeting of the Society in Seattle, Wash., in 1926, a resolution concerning interstate agreements or compacts, was adopted by this Committee, but was presented to the Division too late to be acted upon at that time. This resolution, as amended and adopted by the Division at the Denver meeting, in July, 1927, is as follows:

"*Whereas*, experience has plainly demonstrated that the problem of interstate river controversies, if left to chance solution, leads to costly and protracted litigation involving tremendous economic loss to all concerned; and,

"*Whereas*, efforts to adjust interstate river problems by means of treaties or compacts between the States and the United States Government directly interested promises a more direct and practicable solution of the problems involved;

"*Now, Therefore, Be It Resolved* that the Irrigation Division of the American Society of Civil Engineers definitely recommends that the proper State agencies at the earliest possible date take the initiative in starting and completing such negotiations as will bring about the solution of interstate water difficulties, through treaty or compact, on such broad lines as are best fitted to the individual streams in which such rights may be in controversy."

The tremendous commercial development of this nation within recent years, has forced the Federal Government to take active measures looking to the improvement of its rivers, harbors, and inland waterways, and the development of some of its great water power possibilities.

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* This report has been compiled from two reports published in March, 1928, and March, 1930, *Proceedings*, respectively.

The membership of the Committee of the Irrigation Division on Interstate Water Matters is as follows: M. C. Hinderlider, *Chairman*, George M. Bacon, Frank C. Emerson, A. L. Fellows, C. E. Grunsky, C. S. Heidel, R. I. Meeker, and J. C. Stevens.

Likewise, the pressing demands of irrigation, flood control, power development, and of stream pollution in some of the more thickly populated sections of the East where interstate streams are involved, have accentuated the need for interstate understandings, and have accelerated attempts to solve such problems through interstate compacts or agreements. Such attempts to date have resulted in the ratification of several such compacts and the initiation of studies on many streams interstate in character.

Prior to systematic or efficient development of these great potentialities, it is first needful to ascertain definitely the relative interests of the States affected, and of the Federal Government, in such waters, in order that the reasonable and equitable use of these great resources may be properly conserved for the generations that will live in these various States in the years to come.

Questions of right and necessity arising out of the uses of these natural resources are varied, and, in many instances, most complicated and difficult of adjustment. Controversies respecting international rivers are settled either by treaty or by war. Interstate controversies must be settled by interstate compacts or agreements, or by decree of the United States Supreme Court.

With regard to the sovereign rights of this Nation to withhold from another nation, any of the benefits arising out of the use or enjoyment of the waters of an international stream, the Attorney General of the United States held in substance that the recognition of such a requirement is entirely inconsistent with the sovereignty of the United States over its natural domain; that the United States has a perfect right to divert all the waters of an international stream at any point above an international boundary, irrespective of the effect of such diversion upon the flow of the river in the lower nation, or along that part of its course which may form the boundary between the two nations.

Since the several States of the Union are sovereign in all respects except in so far as they have surrendered or delegated to the Federal Government certain sovereign prerogatives, it might appear that the aforementioned rights of the Federal Government to withhold uses of the water in an international stream, applied with equal force as between the several States; but such is not the case.

With regard to interstate streams, the Supreme Court has heretofore laid down certain general principles applicable to certain conditions, but such decisions fall far short of completely pointing the way in the settlement of all problems interstate in character.

One of the fundamental principles laid down by the Supreme Court is that all States stand on the same level. No State can impose its own legislation on one of the others, and is bound to yield its own views to none.

In one important interstate case the Supreme Court held that "the one cardinal rule underlying the relations of the States to each other is that of equality of right", and in this particular case held that there must be an equitable apportionment of the benefits arising out of the use of an interstate stream.

Upon first consideration it may appear that a settlement of these interstate matters may be determined more expeditiously and effectively by Court procedure than through treaty or compact between the States interested.

This Committee and the Irrigation Division have, through the adoption of the aforementioned resolution, given their approval to the Compact Method as the more desirable means for settling such matters. On the other hand, experience indicates that it would be more difficult to change or modify provisions embodied in interstate compacts, once they have been agreed to, than if the adjustments were the result of a Court decree subject to modification from time to time by the Court having jurisdiction.

While, as stated, it is believed that a compact or treaty is the more logical method for settling questions arising between States, it should be obvious that in such procedure it is essential to approach such questions with ample caution, and that compacts should be signed or ratified only after the most mature deliberation has been exercised in a consideration of the ultimate effect which the same may have upon the future welfare of the peoples, since, when once ratified, such compacts may be practically irrevocable.

The Chairman of the Committee believes that one of the greatest benefits resulting from the application of the compact or treaty method for the settlement of interstate problems, is a desired recognition by the Federal authorities of the principle of State autonomy as opposed to Federal bureaucratic usurpation and control of State resources, a question of growing importance to the Eastern States and one of long standing in the Western States.

The conference on the Colorado River, held in Denver in October, 1927, by the seven interested States, seeking a common basis on which the waters of this stream and the benefits to be derived through the use of the same, may be equitably apportioned, definitely brought to the forefront the paramount question of State *vs.* Federal ownership and control of the natural resources of the West.

While a determination of the relative rights of these States to the use of the water of this great river system is of vital importance, probably it is of less importance to the Western States than the age-old principle of State sovereignty and the inherent rights of these States to control the natural resources therein.

As evidence of the importance of this question, is the so-called Pittman resolution adopted by the Denver Conference of the Governors and Interstate River Commissioners of the Colorado River Basin States, at Denver, in October, 1927, which, on account of the importance, is herein quoted in full:

"*Whereas,* it is the settled law of this country that the ownership of and dominion and sovereignty over lands covered by navigable waters within the limits of the several States of the Union belong to the respective States within which they are found, with the consequent right to use or dispose of any portion thereof, when that can be done without substantial impairment of the interests of the public in the waters, and subject always to the paramount right of Congress to control their navigation so far as may be necessary for the regulation of commerce with foreign nations and among the States, and whereas:

"It is the settled law of this country that subject to the settlement of controversies between them by interstate compact, or decision of the Supreme Court of the United States and subject always to the paramount right of Congress to control the navigation of navigable streams so far as may be necessary for the regulation of commerce with foreign nations and among the States, the exclusive sovereignty over all of the waters within the limits of the several States belongs to the respective States within which they are found, and the sovereignty over waters constituting the boundary between two States is equal in each of such respective States, and whereas:

"It is the sense of this Conference that the exercise by the United States Government of the delegated constitutional authority to control navigation for the regulation of interstate and foreign commerce does not confer upon such government the use of waters for any other purposes which are not plainly adapted to that end, and does not divest the States of their sovereignty over such waters for any other public purpose that will not interfere with navigation:

"Therefore, Be It Resolved, That it is the sense of this Conference of Governors and the duly authorized and appointed Commissioners of the States of Arizona, California, Colorado, New Mexico, Nevada, Utah, and Wyoming, constituting the Colorado River Basin States, assembled at Denver, Colorado, this 23rd day of September, 1927, that:

"The rights of the States under such settled law shall be maintained.

"The States have a legal right to demand and receive compensation for the use of their lands and waters except from the United States for the use of such lands and waters to regulate interstate and foreign commerce.

"The State or States upon whose land a dam and reservoir is built by the United States Government, or whose waters are used in connection with a dam built by the United States Government to generate hydro-electric energy, is entitled to the preferred right to acquire the hydro-electric energy so generated or to acquire the use of such dam and reservoir for the generation of hydro-electric energy, upon undertaking to pay to the United States Government the charges that may be made, for such hydro-electric energy or for the use of such dam and reservoir to amortize the government investment, together with interest thereon, or in lieu thereof agree upon any other method of compensation for the use of their waters.

"We, the undersigned Committee, to which has been referred the foregoing Resolution, as presented to the Conference on August 29, 1927, by Senator Key Pittman, having adopted certain amendments unanimously which are now incorporated therein, recommend that the Resolution set out above be adopted."

Since July, 1927, many important problems concerning interstate streams have arisen. Some of these problems have been settled by Court decision and others by compact, but the solution of the majority of them is still pending. Other important problems are in the offing. The West probably still occupies the center of the arena of interstate water matters, due to the magnitude of the current problems, but during the last two and one-half years several important interstate water controversies have arisen in the East.

Only a few interstate water controversies have been decided to date by the Supreme Court of the United States, the most important of which are: Wyoming *vs.* Colorado; Kansas *vs.* Colorado; and the controversy over diversions by the City of Chicago, Ill., of Lake Michigan water. The first two cases were between States in which the principle of appropriation was in effect. That principle was recognized by the Court, in the Colorado-Wyoming Case. The last, the Chicago Diversion Case, was a controversy between riparian right States, and it was hoped by the States in which that principle applies, that the Court in its decision would establish a precedent by defining the relative rights of riparian owners along a stream system. Such was not the case, the Court basing its decision on the power of the National Government to regulate commerce between States, and more specifically on an act imposing upon the Secretary of War the obligation of removing obstructions to navigation. Two suits have been filed, *viz.*: New Jersey *vs.* New York and Connecticut *vs.* Massachusetts, in the Supreme Court of the United States, which should result in a decision establishing a precedent to be followed in riparian right States. The Special Master in the latter suit has given his opinion which if upheld by the Supreme Court will materially clarify the relative status of riparian right States. He said in effect that the principle of equality of right should be applied and that specific damage must be shown.

Several important international questions over waters are in progress of solution. These are discussed in more detail later.

There is a growing consciousness among the States that have interstate streams, or of which water is an important resource, that the question of Federal *vs.* State control of unnavigable unappropriated waters should be definitely determined.

It is probably true that the principle that States have full control of their waters so long as they are not navigable, is well established, due to the intricate systems of administration and rules of appropriation that States have built up to administer and distribute those waters; the repeated recognition of those systems by decisions of Federal Courts and Executive orders; and the implied recognition of the States' ownership of such resources by various acts of Congress. Yet there are still some attorneys who maintain that the Federal Government has control of interstate streams. The question should be settled. Much discussion has been had on the subject during the last two and one-half years.

The subject is considered of major importance by the Association of Western State Engineers. Since the creation of the Federal Power Commission, New York State has been fighting for the right to control her own water power resources and she has been in a large measure successful. Maine has done likewise, and not only controls her water power resources, but has a law which prohibits the export of power. The law to date has been effective. The States of the South are fighting to control their own water power resources as against Federal control. In a decision by the Supreme Court of the United States over a power case in Wisconsin, the Court pointed out the sovereignty of a State over navigable streams subject to the higher authority of the Federal Government. A certain power company in Wisconsin had failed to comply properly with a law passed in 1925 requiring all owners of power dams to acquire a permit from the State to continue operation, etc. The dam in question was built in 1878 and riparian ownership was claimed. The United States Supreme Court upheld the State law.

It is interesting to note that the Federal Power Commission in its annual report for 1928 advocated legislation in States providing for a joint commission to regulate the transmission of power between the States. If the power were sold direct to consumers the final jurisdiction was to rest with the States, but if the power from one State were sold to distributing companies in another State, the State decisions were to be subject to Federal Court review since such sales would come under the head of interstate commerce.

On the other hand the exigencies of some of the major problems before the country have led some leading engineers to believe that the Federal Government should have control of interstate waters. As an example of this, American Engineering Council's Committee to consider Mississippi Flood Control, composed of John R. Freeman, Past-President, Am. Soc. C. E., and Gardner S. Williams, Baxter L. Brown, and Arthur E. Morgan, Members, Am. Soc. C. E., at the meeting in January, 1928, made the following statement:

> "It is the opinion of this Committee that the authority of the General Government to deal with such questions as are involved in the Mississippi River problem should no longer hang upon such slender threads as the general welfare clause or the regulation of commerce clause of the Constitution, but that the Constitution of these United States should be amended to confer upon the General Government the authority to control and administer the national waters and to assess damages and allocate benefits and costs in connection therewith.
>
> "It is our further opinion that only by the procedure which such an amendment, followed by the passage of a Federal Water Control law, would make possible, can an equitable or justifiable distribution or allocation of the costs of river improvement and flood protection be assumed."

The same view was expressed by Mr. Morgan at the Assembly of American Engineering Council held in Washington, D. C., on January 14 and 15, 1929, and it is understood that steps are being taken, looking to the introduction of the necessary legislation to bring about the proposed amendment.

It is believed from discussion that has taken place in the West that such a procedure as that stated would not meet with favor among the Western States. The Western States prefer to settle their own differences as to interstate waters by compact, or even by Court action when necessary, rather than to submit the administration of those waters to the remote control of a Federal Bureau which would have no knowledge of the complex, intricate problems of a local nature involved in their administration.

It is believed further that the Western States are opposed to submitting control of the upper tributaries of the Mississippi which rise in those States to any Federal Commission—the Mississippi River Commission or otherwise. Such control is gradually being sought on the plea of the necessity of regulating the flow of those tributaries as a part of the Mississippi Flood control plan. The following statement by the Engineering Council Committee cited is worthy of note:

> "Storage reservoirs on the remote head-waters of tributary streams have too limited an effect upon the floods of the Mississippi River to justify their inclusion in the flood protection program for that river."

This discussion is given to indicate the importance and complexity of the problem, and the desirability of full discussion by all interested parties to the end that a constructive program might be evolved for its solution. It is believed that now is an opportune time to bring the problem to the front. The National Executive is an engineer, and it is understood that he is in favor of as little Federal control of National resources as is possible. The Federal Land Commission appointed by him to consider the desirability of turning over to the States, the Government land within those States, and allied matters, should properly consider the all-important question of the control of interstate streams and unnavigable unappropriated waters.

The following gives a brief discussion of some of the most important interstate problems that were in evidence during 1928 and 1929.

Very truly yours,
M. C. Hinderlider, *Chairman,*
Committee of the Irrigation Division
on Interstate Water Matters.

St. Lawrence River

(Canada and United States)

Negotiations have been under way since 1921 between Canada and the United States in an endeavor to reach an understanding concerning the development of the St. Lawrence River as a waterway. In 1921, Messrs. Bowden and Wooten made a joint report to the International Joint Commission concerning the proposed development. In 1924, the United States Government appointed a National Committee called the St. Lawrence Commission of the United States, headed by President Hoover, then Secretary of Commerce. Canada appointed a National Advisory Committee to meet with the United States Committee. These two Committees met and appointed a board of engineers composed of members from the two countries to review the Bowden-Wooten report. The Engineering Board made its report in November, 1926.

The plan of development as finally recommended provides for the construction of a navigation channel 25 ft. deep in the 183-mile section of the river between Lake Ontario and Montreal, Que., Canada, together with nine locks, twenty-five miles of canal, an initial power development of 2 700 000 h.p., and an ultimate development of 5 000 000 h.p.

Some discussion was carried on between the two countries during 1927 and 1928. It is reported that the views of both countries are close together as regards the major features of the project. Canada desires a 27-ft. channel, credit for certain improvements completed on each side of Montreal, and for the Welland Canal, and does not want to develop power beyond her ability to absorb it, as she is opposed to the export of power. It is reported that the United States is of the opinion that Canada should receive credit for only those improvements which will be embodied in the final plan of development, and while the United States Government concedes that all the power developed in Canada and one-half of that developed on the border should go to Canada, yet it believes full development of the potential power possibilities should be made in one stage.

Difficulties arose in Canada over the question of ownership and control of the power that was to be developed by the St. Lawrence Project. The controversy was between the Dominion Government and the Provincial Governments of Ontario and Quebec. The question was referred to the Canadian Supreme Court, but the decision rendered by the Court appears not to have been conclusive. This condition has precluded the possibility of further negotiations between the two Governments pending some sort of agreement between the Dominion Government and the Provinces.

It is reported that Premier MacKenzie King announced in the early part of 1929 that at the close of the session of Parliament a conference would be called between representatives of the Dominion and the Provinces of Ontario and Quebec in an attempt to arrive at an agreement in order that negotiations might be resumed with the United States. It was further stated that no treaty would be finally negotiated until Parliament had ratified each clause.

The New York State Government has recommended the development of power on the St. Lawrence at State expense, the distribution to be made by private interests.

In the meantime Canada has granted the Beauharnois Light, Heat, and Power Company a permit to construct a power canal between Lakes St. Francis and St. Louis, on the St. Lawrence River, to divert 40 000 sec-ft. for the development of an initial output of 500 000 h.p. The permit, however, provides that the canal shall be of sufficient size for navigation, and the whole development is made subject to subsequent additions to make it a part of the St. Lawrence Project should the treaty between the two countries be consummated. Work was commenced on this project in October, 1929.

Niagara Falls

(Canada and United States)

A Special International Niagara Board was created by Canada and the United States in 1925, to investigate the change which was being made in the course of the Niagara River, which it was reported threatened to ruin the beauty of Niagara Falls, and to formulate plans to correct this condition. The Board submitted its report on December 14, 1927, in which it recommended the construction of certain submerged weirs and the enlargement of certain channels to distribute the flow more uniformly over the Falls for the purpose of affecting the desired results and also of providing opportunity for increased diversions.

Following negotiations, a treaty was formulated and a protocol was signed on January 2, 1929, providing for the construction of the remedial works as recommended by the Special Board and for an additional diversion of 10 000 sec-ft. of water on each side of the river between October 1 and March 31 of the following year, for a period of seven years. The treaty of January 11, 1909, provided for a diversion by New York State of 20 000 sec-ft. and by Canada of 36 000 sec-ft.

The ratification of the treaty by the Senate was opposed by the City of Chicago. It is reported that the municipal authorities of Chicago maintain that the treaty should not be ratified until the negotiations for the St. Lawrence Project and the Illinois Waterway are completed, and, further, that the remedial works would lower the lake levels, and the diversion of an additional 20 000 sec-ft. would increase the flow from Lake Erie. This latter view is discounted by Army engineers.

What is said to be considered a victory by New York State in her fight to retain control of power development within the State, was the obtaining from the Niagara Falls Power Company (the beneficiary of the diversion of the additional 10 000 sec-ft. at Niagara Falls) of an agreement to withdraw all objections to rentals now being paid the State for waters, and an agreement on the part of that Company to pay an equitable rental for the use of the additional diversion at Niagara Falls and to seek from the State a license for the use of that water. It is stated that New York considers this a complete recognition of the rights and sovereignty of the State to license power companies and to demand a rental charge for all water now being used and to be used under the treaty.

DELAWARE RIVER

(State of New Jersey *vs.* State of New York and City of New York, Commonwealth of Pennsylvania, Intervenor)

The State of New Jersey has filed suit in the Supreme Court of the United States against the State of New York, to enjoin New York City from making certain diversions from the headwaters of the Delaware River which she has planned for augmenting her water supply. The State of Pennsylvania has filed a motion with the Court to present an intervening petition, and has filed a petition for leave to intervene. The first hearing was set for January 6, 1930.

The plan of development as proposed by New York, provides for a supply of 600 000 000 gal. daily from the flood waters of five tributaries of the Delaware River, and 100 000 000 gal. daily from Rondout Creek, a tributary of the Hudson. This plan provides for the construction of five reservoirs and the necessary aqueducts and appurtenances to bring the water to New York City. The plan also provides for releasing certain quantities of water, when necessary, to maintain a certain specified minimum rate of flow, as was provided for in the last proposed compact. The contemplated development is wholly within the State of New York about 30 miles from the main stream, and affects 600 sq. miles of the total of 2 389 sq. miles of drainage area of the Delaware River in New York. It is maintained by New York that flood waters only are to be stored, and that due to releases from New York reservoirs the low flows will be greater after the development than at present.

The State of New Jersey maintains that it has a right to the undiminished flow of the river, and it is reported that it further alleges that New York State is contemplating the development of power with some of the water diverted.

This controversy is one of several years standing, and it was anticipated that settlement would be made by compact. A compact was negotiated in 1925, was signed by the Commissioners appointed by the three States, and was ratified by the State Legislature of New York, but failed of ratification in New Jersey and Pennsylvania. Another compact was formulated in 1927. It was signed by the representatives of the three States, but again failed to be ratified in Pennsylvania and New Jersey although it was ratified by the New York State Legislature.

The City of Trenton, N. J., was one of the principal objectors to the compact in New Jersey.

After failure to consummate a compact, New York City retained Attorneys Hughes and Davis to give their opinion as to the right of New York to make the proposed diversion. Both attorneys held in effect that if the facts were as they were purported to be, the Supreme Court would probably allow the diversion and provide for the payment of damages, if any, to lower interests; whereupon New York decided to proceed with the development, which action precipitated the suit.

It is believed the following portion of Mr. Hughes' opinion is of particular interest in the light of growing importance over the country regarding Federal *vs.* State control of unappropriated non-navigable waters. Mr. Hughes says:

> " 'Whether Congress has power to provide the diversion of navigable waters from one water-shed to another is a question which may be regarded as not yet finally determined' if 'Congress has this power it would be exercised only in interest of interstate commerce, that is, navigation.' Similarly the Secretary of War could not act except to protect navigation. Therefore unless the proposed diversion would injuriously affect navigation, 'the United States will not be entitled to complain.' "

The taking of testimony started on April 21, 1930, at Trenton by Charles N. Burch, Special Master.

The outcome of the suit should be of great importance to riparian right States. Many future water supply developments in the East must be along lines similar to those proposed by New York, that is, involving riparian rights in two or more States.

CONNECTICUT RIVER

(State of Connecticut *vs.* Commonwealth of Massachusetts and Boston Metropolitan Water District)

The State of Connecticut filed a suit in the Supreme Court to restrain the City of Boston, Mass., from diverting water from the Swift and Ware Rivers, tributaries of the Connecticut River. Charles W. Bunn was appointed on December 2, 1929, by the Supreme Court as Special Master to take testimony.

In May, 1930, Mr. Bunn in his report upheld the right of the City of Boston to construct the proposed project and held that the damage that will result to navigation, to agriculture, and to the sanitary condition of the Connecticut River by the project will be inconsequential. He recommended the payment of a certain sum for damages that will result to a certain proposed water project if and when the project is constructed.

Mr. Bunn's report is noteworthy in that he held against the principle as set forth by Connecticut that any invasion of the rights or privileges of a State calls for an injunction regardless of damage. Although both States have adopted the doctrine of riparian rights, Mr. Bunn held that the principle of equality of right should be applied and that specific damage must be shown to justify an injunction.

Lake Michigan

(Pollution of Chicago Water Supply. Chicago and Indiana Towns on South Side of Lake)

Negotiations are under way between the City of Chicago and certain Indiana towns on the south side of Lake Michigan to provide for the elimination of the pollution of the Chicago water supply by phenol wastes from coke-oven plants in the Indiana towns. The pollution occurs when unusually strong winds blow from the south carrying the phenol across the lake in the vicinity of the intakes of Chicago's water supply system.

As an example of the amount of the pollution, it is reported that during the occurrence of strong winds in the latter part of 1927, it required 13.6 lb. of chlorine per 1 000 000 gal. for purification purposes while the normal quantity of chlorine necessary is 2 lb. per 1 000 000 gal. The excessive quantities of chlorine and phenol cause the water to have a bad taste.

It is reported that plans are already under way or completed for the elimination of the phenol nuisance in the Calumet and Michigan City Districts.

Chicago Diversion from Lake Michigan

The United States Supreme Court by a decision rendered on January 14, 1929, ended a 20-year controversy between the City of Chicago and all the Lake States, except Illinois and Indiana, regarding certain diversions that the city has been making from Lake Michigan for sewage dilution and other purposes. The controversy finally involved thirteen States, practically all the Mississippi River States supporting Chicago on the plea that the diversions aided navigation along the Mississippi. The Supreme Court held in its decision that the diversion as made by Chicago was illegal; that no diversion was legal except in the interest of navigation; that the increase in diversion made by Chicago from 4 167 sec-ft. to 8 500 sec-ft. defied the authority of the National Government resting in the Secretary of War; and that Chicago must proceed with dispatch to remedy the condition. Provision was made for further hearings to determine the minimum quantity of water necessary to be diverted in the interests of navigation, and to determine the length of time that should be allowed the City of Chicago to reduce the diversions to that point.

Hearings to determine these questions were held before the Hon. Charles E. Hughes, Master, during 1929. Mr. Hughes submitted his report to the Supreme Court on December 17, 1929, in which he specified a certain program of construction of sewage treatment works that Chicago should follow to enable the diversion from the lake to be reduced from its present amount of 8 500 sec-ft.

to a permanent amount of 1 500 sec-ft., on December 31, 1938. Mr. Hughes also recommended that Chicago should be required to install control works to prevent the reversal of flow through the Chicago River during storms; the plans for the control works to be approved by the Chief of Engineers of the War Department; and those works to be completed within two years after receiving the authorization of the War Department.

On April 14, 1930, the Supreme Court approved substantially the report of Mr. Hughes.

The plaintiffs in the case alleged that the diversion of water as made by Chicago from Lake Michigan interfered with navigation on the Great Lakes by lowering the surface of the lakes by as much as 6 in.

The carrying out of the provisions of the decision of the Supreme Court will necessitate, probably, the greatest expenditure of money ever made to conform to a Court decision regarding water matters. Chicago engineers estimate that the works necessary to carry out the Court order will cost $176 000 000. However, the Court decision was not important from the standpoint of precedent. It did not determine relative rights of riparian owners which is an important question in the East. The decision was based upon an act passed by Congress in 1899, which gave to the Secretary of War and the Chief of Army Engineers authority to prevent obstruction to navigation. However, the decision did state that Congress "may not arbitrarily destroy or impair the rights of owners by legislative action which has no real or substantial relation to the control of navigation or appropriateness to that end."

Rio Grande, Colorado, Tiajuana

(United States and Republic of Mexico)

Probably the most important and involved question concerning international streams in the United States is that of allotment of waters of the Rio Grande (from Fort Quitman, Tex., to the Gulf of Mexico), Colorado, and Tiajuana Rivers, between the United States and Mexico.

A large body of land is irrigated in Texas along the Lower Rio Grande. Several millions of dollars are now being expended in the further development, and more development is contemplated. Most of the water originates in Mexico and comes to the Rio Grande through tributaries, the main ones of which are the San Juan and Salado. There are large areas of land in Mexico susceptible of irrigation from those streams. Development in Mexico is far behind that in the United States, but some development on a large scale has been started. The Don Martin Project being constructed by the Mexican Government will provide for the irrigation of about 165 000 acres of land from the Salado. A treaty is necessary to provide for a proper allocation of the water supply, or future Mexican uses might encroach upon present uses being made of the waters by American land owners.

On the Colorado River conditions are the reverse of those on the Rio Grande. The water supply originates in the United States, whereas there is a considerable acreage of land in Mexico being watered by the Colorado River which might be deprived of its water supply by future uses in the United States unless an allocation is made by treaty.

Tiajuana River meanders back and forth across the western end of the international boundary between the United States and Mexico. The City of San Diego, Calif., and water users in that vicinity need Tiajuana water; so do Mexican users. Due to the character of flow, storage is required. The Mexican Government is now building a reservoir for that purpose. An allocation of those waters should be made.

A commission was appointed by the United States, composed of Elwood Mead and Lansing H. Beach, Members, Am. Soc. C. E., and Mr. W. E. Anderson, to meet with a similar commission representing the Republic of Mexico, to discuss these matters and devise a basis for settlement. The Mexican Commission was appointed in 1927, while the American Commission had been in existence for some time.

The Joint Commission was very active in 1929, several meetings having been held. More are scheduled for 1930. No report has been made public as to progress, but it is hoped the basis for an early settlement has been reached.

For some time the Rio Grande between El Paso, Tex., and Fort Quitman has been under discussion by the International Boundary Commissions of the United States and Mexico. The questions involved are flood control and means to prevent channel changes which have caused some difficulty in determining ownership of certain tracts of land. Considerable time was spent by the two Commissions in making detailed investigations and apparently an early settlement is in view. The American International Boundary Commission made a report to the Secretary of State in June, 1929, on the engineering feasibility of a plan to solve the problems involved.

Rio Grande

(Colorado, New Mexico, and Texas)

A series of conferences were held between these States at Santa Fé, N. Mex., beginning in December, 1928, and ending in February, 1929, for the purpose of formulating a compact providing for the allocation of the waters of the Rio Grande. The conferences were presided over by Col. W. J. Donovan, representing the United States.

It developed during the negotiations that it would be impossible at that time to draft a compact which would make a definite allocation of water satisfactory to all the interested States, due principally to the fact that the ultimate water supply of the basin which was to be divided, was not known.

Of the water-shed of the Rio Grande in Colorado 2 900 sq. miles constitute a closed basin that is not tributary to the Rio Grande, but may be made so by the construction of an outlet drainage canal tapping the area. Such canal would materially augment the quantity of water contributed by Colorado to the Rio Grande.

A temporary compact was formulated and signed by the Interstate River Commissioners for the respective States, providing for a condition of *status quo* on the river for a period of five years, during which time concerted effort would be made by the three States to bring about the construction of the drainage canal from the closed basin in Colorado, together with the construction of a storage reservoir at the Colorado-New Mexico State line, all at Government expense. After the construction of these works, or at the expiration of five years, further negotiations are to be entered into for the consummation of a permanent compact.

The compact provides for the construction of these works by the Federal Government. The claim of the three States upon the Federal Government is predicated upon the fact that some years ago the United States Government entered into a treaty with the Republic of Mexico whereby there was ceded to Mexico in perpetuity 60 000 acre-ft. per annum from the waters of the Rio Grande. This treaty was not the result of any legal obligation upon the United States, but was in the interest of international comity. In an opinion rendered at the request of the Department of State, Attorney General Judson Harmon said in regard to the validity of Mexican claims:

> "Whether the circumstances make it possible or proper to take any action from considerations of comity is a question which does not pertain to this department; but that the question should be decided as one of policy only, because the rules, principles, and precedents of international law impose no liability or obligation upon the United States."

The treaty specifically states that Mexican claims were not conceded and it stipulated that the settlement was not to be construed as a precedent.

The three States contend that the 60 000 acre-ft. which were ceded to Mexico were taken from the common water supply of the three States for the benefit of the whole Nation (to preserve international comity and peace) and, therefore, the cost of construction of the outlet drain and reservoir provided for in the compact, and which will result in returning to the States the equivalent of the water ceded to Mexico, is an obligation of the United States Government.

The compact as formulated was ratified by the Legislatures of the three States and by Congress. A bill has been introduced in Congress providing for the construction of the outlet drain as provided for in the compact.

Colorado River

(Arizona, California, Colorado, Nevada, New Mexico, Utah, and Wyoming)

Progress of Negotiations Since July, 1927.—In an effort to reach an agreement on the division of the waters of the Colorado River allotted to the Lower Basin States by the Santa Fé-Colorado River Compact of 1922, further conferences were held between representatives of Arizona, California, and Nevada during 1927, but to no avail, the major difficulty being a difference of opinion as to the proper allotment of water and of benefits derived from power that might be developed along the Colorado River.

The Swing-Johnson Bill, after many amendments and vicissitudes, passed the House of Representatives in June, 1928, and the Senate during the session of 1928-29. The bill as passed is termed the Boulder Canyon Project Act and among other things provides for the following:

1.—Construction of a dam at Boulder or Black Canyon of not less than 20 000 000 acre-ft. capacity to control floods, improve navigation, store water for use in the United States, and to generate power.

2.—An appropriation of $165 000 000 to construct the project, of which $25 000 000 was designated for flood control, the $25 000 000 to be replaced out of 62½% of any revenue in excess of the amount necessary to repay the Government for the remainder of the cost.

3.—That contracts shall be made by the Secretary of the Interior for the sale of power, and water in sufficient amount to replace the Government investment in the dam and power plants in fifty years, before construction is started, the power to be sold at a price "justified by competitive conditions at distributing points or competitive centers."

4.—That 37½% of any moneys collected by the Secretary of the Interior above the amounts due the Government, shall go to Arizona and Nevada by virtue of their natural resources being taken for a public service.

5.—Allotment of 300 000 acre-ft. of water to Nevada and 2 800 000 acre-ft. to Arizona for exclusive beneficial and consumptive use in perpetuity. That Arizona may annually use one-half of the excess or surplus waters unapportioned by the Colorado River Compact and all the Gila River.

6.—That the Act would become effective when seven States ratified the Colorado River Compact, but, failing this, it would become effective by proclamation of the President six months after passage, if six States had ratified the Compact.

7.—All provisions of the Act and the future operation of the project were made subject to the Colorado River Compact.

The Act also provided that the All-American Canal may be constructed, the cost to be paid by the lands benefited.

The Act was approved December 21, 1929. Arizona did not ratify the Colorado River Compact, but Utah did so in March, 1929, which provided the necessary six States. In June, 1929, President Hoover declared the Act effective.

When notice was given that bids would be received for power, the applications were greatly in excess of the available power. After the Secretary of the Interior made his announcement of the tentative allocation of power and the price to be paid for it, some of the States, notably Arizona and Nevada, took exception to such allocation. Pending a further effort to reach an agreement between the Lower Basin States, the Secretary announced that no contracts for power or water would be let immediately, if Arizona, Nevada, and California would enter into further negotiations in an attempt to reach an agreement. Arizona had prepared a suit to be filed in the United States Supreme Court. The request of the Secretary was complied with, and Arizona refrained from filing her suit pending the outcome of conferences between the Lower States to start January 20, 1930.

The Lower States' Conference was held at Reno, Nev., but adjourned without an agreement having been reached. A controversy which arose at the conference between California interests over the proper division of the water allotted to California was later settled when it was agreed that Imperial Valley should receive 3 850 000 acre-ft., and the Metropolitan Water District, 550 000 acre-ft. of Class A water. From the unappropriated surplus the Metropolitan Water District is to receive an additional amount up to 550 000 acre-ft. Any surplus over 1 100 000 acre-ft. allotted to California should be given to the Imperial Valley.

The power contracts were signed April 26, 1930. The allotment of power was as follows: Primary power, Arizona, 18%; Nevada, 18%; Metropolitan Water District, 36%; Los Angeles, 13%; eleven municipalities in California, 6%; and the Southern California Edison Company and associated companies, 9 per cent. The Metropolitan Water District is to use all the secondary power it has need for, the remainder to be divided equally between Los Angeles and a group of four power companies. The specified present price for power at the switchboard to lessees is 1.63 mills per kw-hr. for primary power, and $\frac{1}{2}$ mill for secondary power, and to allottees who are not lessees the price is 2 mills per kw-hr. for primary power and $\frac{9}{10}$ mill for secondary power.

On June 24, 1930, an injunction suit was filed in the Supreme Court of the District of Columbia by Fred T. Colter as Trustee of the Arizona Highline Reclamation Association to restrain the Secretary of the Interior from taking any action under the contracts with the City of Los Angeles and the Metropolitan Water District.

An appropriation of $10 660 000 has been made by Congress to start work on the project.

The Interstate Water Commissions of the Upper Basin States—Colorado, Wyoming, New Mexico, and Utah—held a preliminary conference in Denver, Colo., on December 5, 1929, for the allocation of the 7 500 000 acre-ft. of water set aside for the Upper Basin by the Colorado River Compact.

Due to the fact that some of the States did not have all their data prepared, the Conference adjourned to be reconvened at such time when all States were ready. From the preliminary discussion it is not anticipated that any great difficulty will be experienced in consummating a compact among these four Upper Basin States.

Laramie River

(Colorado and Wyoming)

Discussions have taken place between the States of Colorado and Wyoming with respect to present uses of water in Colorado from the Laramie River, an interstate stream.

Some years ago as a result of a suit instituted by the State of Wyoming against the State of Colorado, the Supreme Court of the United States rendered a decision relative to the uses of the waters of this stream by the two States.

It appears from these negotiations that Colorado places a materially different interpretation on the decree of the Court than the State of Wyoming, which may require a construction of the decree by the Court to clarify the present situation.

La Plata River

(Colorado and New Mexico)

In 1922, Colorado and New Mexico ratified a compact providing for the division of the waters of the La Plata River, a small interstate stream heading in Southwestern Colorado and flowing into New Mexico. This compact is administered by the State Engineers of the States in question.

The provisions of the compact permit rotation of water between the two States during times of shortage. This practice resulted in a suit instituted by one of the water users in Colorado for the purpose of enjoining the water officials of Colorado from continuing such practice, which it is alleged results in injury to the plaintiff. This suit has been watched with interest since in reality it is a direct attack upon the validity of the compact which was the first one negotiated by Colorado with her sister States with respect to the use of waters of interstate streams. The District Judge before whom the case was tried recently entered a decree in favor of the State.

It should be borne in mind that the aforementioned suit is not of an interstate or international character, but at present is an internal affair.

North Platte

(Colorado and Wyoming)

Negotiations have been carried on during the past two years between Colorado, Wyoming, and Nebraska with respect to the use of the waters of the North Platte River, which flows through the three States.

The Federal Government, through the Bureau of Reclamation, appears to be an interested party in this connection as a result of the construction of the Pathfinder Reservoir in Wyoming, and later of other reservoirs and canal systems in Eastern Wyoming and Western Nebraska. These Federal projects have an important bearing upon the equalization and administration of stream flow in Wyoming and Nebraska.

It is confidently anticipated that a successful termination of these negotiations will be had at a comparatively early date.

Arkansas River

(Colorado and Kansas)

Litigation between the States of Colorado and Kansas over the waters of the Arkansas River has been practically continuous for the past twenty-eight years. In the noted suit brought by Kansas against Colorado in 1901 the United States Supreme Court decided that although certain irrigation interests in Western Kansas had been injured by diversions of water from the Arkansas River in Colorado, that such diversions in that State had not exceeded its equitable share of the waters of this interstate stream, dismissed the case without prejudice against Kansas to re-institute suit at a later date if continued diversions in her judgment justified such action. Kansas has not elected to do so to date.

Almost immediately after the settlement of that suit, certain ditches in Kansas instituted suit in the Federal Court for the District of Colorado against a large number of ditches and reservoirs in Colorado, seeking to restrain the latter from diverting water in Colorado to their alleged injury in Kansas. The first of these suits was settled out of Court, but a later suit was instituted in the same Court by one of the Kansas ditches not a party to the former settlement. This condition resulted in Colorado in her sovereign capacity filing suit in 1929 against the State of Kansas for the purpose of requiring the latter State to enforce the provisions of the former Supreme Court decree and quiet further litigation by her agents. Testimony in this important case is now being taken before a Commissioner of the Supreme Court.

Negotiations between the Interstate River Commissioners of the two States for some years past looking to the formulation of an interstate compact with respect to the uses of the water of this river, have to date failed in their objective and, as a result of the present suit, have been entirely discontinued.

KOOTENAY RIVER

(Canada and Idaho)

Owners of reclaimed lands in Northern Idaho objected in the fall of 1929, to the proposed plan of the West Kootenay Power and Light Company of British Columbia, to construct a dam in Kootenay River at Granite, Idaho, on the plea that such a dam would cause the flooding of their lands. The development as proposed by the Power Company is for the purpose of providing additional power.

The International Joint Commission authorized the Company to make the necessary excavations for the dam, but stated that final decision would be withheld until data relative to possible damage were secured.

COLUMBIA RIVER

(Idaho, Montana, Oregon, and Washington)

In December, 1924, the Governor of the State of Washington, addressed a communication to the Governors of the States of Oregon, Montana, and Idaho urging the necessity of a compact for the allocation of the waters of the Columbia River and its tributaries, and the appointment of an allocation board to consider the matter. A similar request was addressed, at the same time, to the Secretaries of War and the Interior. As a result of these communications the Governors of the respective States and the Departmental Officials designated representatives to serve in such capacity. The more controversial issues on this stream appear to be between Washington, Idaho, and Montana in reference to storage of water in the upper two States of Idaho and Montana for use in the State of Washington. Immediate action was taken by these representatives looking to the formation of a compact between the interested States, which has been sanctioned by the Federal Government.

Conferences have been held for the purpose of considering an early draft of a proposed compact submitted by the representatives of the State of Washington and for studying the manifold phases of this problem. Due to the involved nature of the subject and the magnitude of the problem, no definite conclusions appear to have been reached at this time. Such studies have been very largely limited to Clark Fork, Pend' Oreille, and Spokane Rivers and their tributaries, and to storage possibilities in Priest, Pend' Oreille, and Cœur d'Alene Lakes.

The State of Washington is interested primarily on account of the proposed Great Columbia Basin Irrigation Project in that State. Idaho and Montana are vitally interested in the preservation for future use of sufficient quantities of the waters which rise in those States to assure the legitimate development of their great natural potentialities.

Complicated problems involving power development, navigation, storage, private and corporate interests, etc., are involved. The Federal Government is also interested in any compact allocating the waters in the Columbia River, since certain reclamation projects may be involved, and since also the main river is international in character.

During 1925 and 1926 the State of Idaho made some study of the proposal offered by the State of Washington, but due to lack of definite information as to quantities of water required by the State of Washington and other related matters, the representative of the State of Idaho declined to enter into a compact at that time.

During 1925 and 1926 rather extensive investigations and studies were carried on by the State of Idaho under the direction of W. G. Swendson, Assoc. M. Am. Soc. C. E., former Commissioner of Reclamation of that State, the results of which are set forth in a comprehensive report.

The State of Washington has recently filed with Idaho and Montana a more definite plan of the proposed operations in that State as affecting the use of the waters of the tributaries of the Columbia River. The Committee appointed by the Allocation Board has held numerous conferences as a result of which a tentative draft of agreement on matters concerning Washington, Montana, and Idaho has been prepared.

The principal issue between Washington and Idaho appears to center about the use of Lakes Pend' Oreille and Cœur d'Alene for stream regulation. The owners of property rights about these lakes have raised serious objections against any regulation of the lakes which might be detrimental to their interests.

Similar negotiations in this connection are proceeding between the States of Washington and Montana, with the understanding that progress reports in this regard will be forthcoming at an early date.

SNAKE RIVER

(Idaho, Oregon, Washington, and Wyoming)

Congress has authorized the negotiations of a compact between the States of Idaho, Oregon, Washington, and Wyoming, for the allocation of the waters of the Snake River. It is understood that the initial steps in this matter were taken by Wyoming. From latest reports, negotiations in this regard are inactive, the other States concerned having taken no steps, pending further action by Wyoming.

It is understood that the Commissioner of Reclamation of the State of Idaho has requested an allocation of funds with which to take care of an investigation on this stream. At present, the Federal Government is not involved in the proposed compact.

St. Mary-Milk River

(Montana, United States, and Canada)

It is understood that steps have been taken to re-open the St. Mary-Milk River case before the International Joint Commission on the ground that under the order of 1921 the United States received less than one-half the water of this stream in 1927, whereas it is contended that the treaty of 1909 should provide an equal division of the waters between Canada and the United States.

Records of stream flow are being obtained at a number of new gauging stations for the purpose of providing additional information essential to a proper consideration of this case.

Carson-Virgin-Owyhee Rivers

(Nevada, Utah, Idaho, and California)

The State Engineer of Nevada reports that negotiations concerning the waters of these interstate streams may be initiated at a later date.

James River

(North Dakota and South Dakota)

The State of North Dakota is endeavoring to interest the State of South Dakota in an agreement looking to storage of the run-off from the James River in North Dakota for increasing the low-water flow on that stream.

North Dakota also desires to divert a portion of the flow of the Missouri River to the eastern part of that State, and improve conditions at Devils Lake and on the Cheyenne and James Rivers, for the purpose of supplementing the present run-off and providing domestic water supplies for the cities along these streams.

South Platte River

(Colorado and Nebraska)

The compact was ratified and its administration by the State Engineers of the two States was initiated in the spring of 1926. The results to date are most satisfactory.

Canadian River

(New Mexico, Oklahoma, and Texas)

On December 31, 1926, the Commissioners for the States of New Mexico, Texas, and Oklahoma, signed an agreement for the control of the Canadian River unit in the interstate control of the Arkansas River System.

This agreement provides for the control of the flood waters of the Canadian River and the apportionment of the use of such waters as well as the equitable apportionment of the cost of control according to benefits received by the several States interested. Other features of the agreement establish the relative importance of the different beneficial uses of the water, promote interstate comity and friendly relations and mutual and common developments, remove causes of present and future controversies, and secure expeditious protection from said river and the expeditious agricultural and industrial developments of the Canadian River, the storage and use of its waters, and the protection of life and property from flood. It is anticipated that the State of Arkansas, also a party to these negotiations, will become one of the signatory States in this connection.

This agreement voices the hope that the States of Colorado, Kansas, Louisiana, and Mississippi may subscribe for the complete control of the entire Arkansas River System.

OHIO RIVER

(Illinois, Indiana, Kentucky, New York, Maryland, Ohio, Pennsylvania, Tennessee, and West Virginia)

One of the outstanding examples illustrating the effective application of the principle of interstate compacts is furnished by the agreement entered into between the States of Illinois, Indiana, Kentucky, New York, Maryland, Ohio, Pennsylvania, Tennessee, and West Virginia for the prevention of pollution of waters of the Ohio River and its tributaries through control of phenol wastes.

The pollution of the waters of the Monongahela, Beaver, and other tributaries of the Ohio River, had become so acute in the three States of Pennsylvania, Ohio, and West Virginia, that it appeared imperative to take some concerted action looking to a correction of the condition; and since the Ohio River is an interstate stream, corrective effort had to be applied along streams originating in different States, and, hence, stream pollution involves interstate negotiations.

The first step in this most important piece of interstate diplomacy was initiated by Surgeon-General Cummings, of the U. S. Public Health Service, in 1923, at the instigation of the Ohio State Health Department, by calling a conference in Washington, D. C., at which representatives of fifteen State Health Departments and the U. S. Bureau of Mines, U. S. Bureau of Standards, and the U. S. Public Health Service, were present. This was followed by a meeting of representatives of the manufacturers of by-products of coke, with the Ohio State Health Department, at which time an agreement was reached to keep the phenol wastes out of the streams of that State.

Early the following year (1924) the Health Departments of the States most interested met in Washington where an organization of the Health Departments of the States on the Ohio River water-shed was effected. In April, 1924, at a meeting in Pittsburgh, Pa., the State Health Departments most interested held a conference with representatives of all the by-product manufacturers from the several States, at which time joint similar policies for co-operating States to carry out, were adopted. In November, 1924, the Health Departments of Pennsylvania, Ohio, and West Virginia, through their Health Commissioners, signed an agreement respecting the control of phenol waste disposal. The success of such an agreement has since led to the formation of plans by the nine States tributary to the Ohio River for the regulation of the pollution of other streams in this system.

The original group of three States has been enlarged to include the nine States mentioned, which have become signatories to this interstate compact known as The Interstate Stream Conservation Agreement. This agreement, affecting the health and lives of 15 000 000 people, is a monument marking the progress of constructive statesmanship, and as a result of such success, it is reported that the cry for "Federal control of stream pollution" is becoming less insistent. Dr. F. H. Frost, of the U. S. Public Health Service, is quoted as having stated that this concerted effort of the States co-operating with industry on a common policy, gives infinitely more promise of success than the enactment of Federal laws to regulate the pollution of interstate streams; and, hence, another menace to State autonomy has been eliminated.

DISCUSSION

R. I. Meeker,* M. Am. Soc. C. E. (by letter).—This report is a valuable summary of the status of interstate water controversies confronting one-half the States of the Union, and of international river problems under consideration by the United States. A glance at the various headings is sufficient to show the importance, imminence, and widespread character of interstate river controversies, including those of power, pollution, drainage, flood control, and irrigation uses, with inter-water-shed diversions for municipal supplies, power, and sewage dilution. The problems involve navigation, power, irrigation, and flood control. The writer is in full accord with the prefatory statements of the report.

In the Arid West water has long been considered the critical resource, and the limit of future development is measured in terms of potential water supplies. Interstate rivers are a common resource, and State equities are subject to and limited by the needs and rights of the various interested States. A keen awakening is apparent in the natural units of river basins and among States as to the importance of defining future usable water supplies; hence, the competition and contests for water now under way to secure and define the rights in interstate rivers. Such action is logical. State titles to interstate rivers should be settled in advance of further utilization, in order that delays to development and financial losses may be avoided.

Almost without exception the Western States are spending considerable sums of State moneys to determine the facts as to their unused water supplies. This is especially true of Nebraska, Colorado, New Mexico, Utah, and California. California's marvelous growth is a direct result of utilization of its rivers for power and irrigation. While its rivers produce annually 75 000 000 acre-ft. of water, the water supplies of Southern California are meager and limited. Hence, the need for Colorado River water and the recent purchase by the City of Los Angeles of Mono Basin water which will be taken through an 11-mile tunnel to Owens Valley where diversion can be made by the present aqueduct. The Gila River in Arizona soon will be entirely consumed by irrigation and power uses. Its waters are involved in the Colorado River controversy. A water compact on the Gila between New Mexico and Arizona is inevitable, and is imminent. To-day, the Arkansas River is 85% consumed at the Colorado border. Since 1901 interstate titles to this river have been in dispute and are still unsettled between the States of Kansas and Colorado. Partition of its waters is correspondingly difficult at this late date because of attendant property values created by the use of its waters. The lesson to be learned is that interstate river titles should be settled while water utilization is in its infancy and before large property rights have been created by use of its waters. Conditions are then favorable to division by agreement without the interminable bickering and haggling always attendant upon ownership of water already put to beneficial use.

The South Platte River is 85% consumed by irrigation uses in Colorado. Fortunately, Nebraska's demands on this river are relatively small and permitted of an early interstate settlement by compact in 1923. Another example of an interstate river on which heavy development and consumption had taken place before settlement of interstate titles, is the Rio Grande above Fort Quitman, Texas. Large areas are irrigated in the States of Colorado, New Mexico, and Texas. The unused water discharging from the lower end of the El Paso Valley at Fort Quitman, is only 13% of the basin water supply originating above that point. Consumption above Fort Quitman is, therefore, 87% of the basin water supply of 2 260 000 acre-ft. In 1930, the three interested States of Colorado, New Mexico, and Texas perfected a preliminary interstate agreement concerning the waters of the Rio Grande, a difficult task when consideration is given to water claims, present and future, of citizens in three States, with water consumption practically 90% of the basin supply.

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* Cons. Engr., Denver, Colo.

It should be obvious that interstate allocation, whether by Court procedure or compact, should be settled before development has crystallized too deeply and water consumption has advanced to the stages outlined on the Arkansas, South Platte, Gila, and Rio Grande.

The dominant principle underlying interstate river compacts has been the settlement of water titles in advance of large created property values; also the protection of future development in portions of river basins where economic conditions do not warrant utilization at the present time. Water appropriations with priorities invariably follow economic feasibility in the irrigated West. The doctrine of priority is limited to intrastate scope by interstate river compacts. State allocations provide both for present and future development. The virtue of the Colorado and Canadian River compacts is that water utilization and consequent consumption are still in their infancy in those river basins.

With reference to interstate river decisions, the case of Missouri *vs.* Illinois was omitted from the Committee report. This controversy was litigated in the Supreme Court of the United States. The primary matter in dispute was the pollution of the Mississippi River by sewage from the Chicago Drainage Canal. The gist of the decision was that insufficient injury was shown, and the case was dismissed.

Mention should also be made of Nebraska *vs.* Colorado, litigated in the U. S. District Court of Colorado and carried to the Supreme Court of the United States. This interstate dispute concerned the waters of the Republican River and their use through an interstate canal serving lands both in Colorado and Nebraska. The decision in substance held that Colorado water officials must permit waters to pass to the Pioneer Canal for the irrigation of lands in Nebraska, thus recognizing an interstate priority. The first suit was filed in 1914, reached the U. S. Supreme Court in 1917, and was affirmed in 1923.

The writer is especially interested in that phase of interstate rivers dealing with Federal and State titles. Having lived in the irrigated West for thirty years where the States own, control, and administer their rivers, the writer is unable to find himself in accord with the Eastern viewpoint, drafted by a committee of prominent engineers of Engineering Council, concerning Mississippi River flood control. This group proposes an amendment of the Constitution of the United States, whereby the Federal Government will be given authority and control to administer the National waters. The States, and not the Nation, are superior regarding legislation concerning the use of public waters, except only in the matter of navigation, wherein the Nation controls. While it would undoubtedly be convenient to have complete Federal authority on some rivers, in order to effect flood control more readily, it is not necessary, and is viewed as a step in the wrong direction. Long-range control and administration of rivers by officials in Washington would not be countenanced by the Western States. The States have responsibilities and obligations that should be faced and met frankly and fairly at State expense without 100% recourse to the Federal coffers. Nearly every acceptance of Federal moneys results in a yielding of State autonomy. "Let Uncle Sam do it" is becoming a favorite pastime to the great injury of local government. At present, there is a decided tendency by various tributary areas in the Mississippi River Basin to seek Federal funds for purely local benefits under the guise of flood-control benefits to the Lower Mississippi Valley.

M. C. HINDERLIDER,* M. AM. SOC. C. E. (by letter).—It was the hope of the writer as Chairman, and of the other members of the Committee, that its progress report on Interstate Water Matters would elicit more discussion. Since the viewpoint of the Committee in regard to interstate waters is the viewpoint of the Western Arid States where water is a principal asset and where the principle of appropriation, in general, is in effect, it was the desire of the Committee that in the discussion of the report the viewpoint of the Eastern States, where the riparian doctrine is applied to waters, would be expressed, particularly since several very important controversies over interstate waters are in the process of solution in that region.

It was also the hope of the Committee that there would be some discussion in regard to Federal *vs.* State Control of interstate waters, in which the viewpoint of those States where flood control is a major problem would be expressed.

The Committee is grateful to R. I. Meeker, M. Am. Soc. C. E., for his very excellent discussion of its report, and appreciates his calling attention to the omission from the report of two important interstate controversies that were settled in Federal Courts.

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* State Engr. of Colorado, Denver, Colo.

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DRAINAGE OF IRRIGATED LANDS

REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION*

WITH DISCUSSION BY M. R. LEWIS, M. AM. SOC. C. E.

HISTORICAL OUTLINE

The necessity for providing drainage to remove water from irrigated lands seems anomalous. After paying the cost of supplying water to land, why should it be necessary to spend more money to take away a part of this water?

The earliest records of activities of the human race in irrigation show that the drainage problem has always been important and lack of drainage has been a factor in great historical events. The Valley of the Tigris and Euphrates Rivers in old Mesopotamia, to a large extent, has returned to the desert. At one time nearly 10 000 000 acres of land "in ancient Chaldea" were, to quote the late Sir William Willcocks, "as fertile as a garden." The greater part of this region now consists of alkali flats and saline areas, barren of all but meager feed for flocks and herds of wandering tribesmen. Lack of drainage no doubt played a large part in causing the people of this region to fall behind in the struggle for existence. Arabia and the Sahara Desert are dotted with the remains of abandoned irrigation works, saline lands, salt-marsh areas, and abandoned fields showing signs of former water-logging and accumulations of alkali. Egypt is now engaged in reclaiming from alkali areas which in the days of the Ptolemys were known as the most fertile along the Nile. The land of Goshen, to the east of Cairo, assigned the Children of Israel during their sojourn in Egypt, is now being brought back into cultivation after it had been abandoned to alkali for many generations. In India, drainage problems for many years threatened to wreck the irrigation projects fostered by the British.

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* Published in March, 1928, *Proceedings*.

The membership of the Committee of the Irrigation Division on Drainage of Irrigated Lands is as follows: Thomas H. Means, *Chairman*, J. L. Burkholder, L. M. Holt, H. S. Kleinschmidt, and L. M. Lawson.

Even in America, historical facts point to the drainage problem as influencing the activities of early irrigators. The prehistoric irrigators in the Salt River Valley of Arizona must have been troubled with alkali, for along the ancient ditches alkalied areas have been found in many places, indicating that drainage problems had an important part in the lives of these little known irrigators.

The first of the modern tribe to practice irrigation, the Mormons, found drainage an essential part of their irrigation works. They were many years in learning the remedy but, to-day, Utah has carried on more extensive drainage than any other State. The U. S. Reclamation Bureau has been beset with the necessity for continuously extending drainage works and adding new drainage to areas where none had been planned at the time the area was first watered.

The U. S. Department of Agriculture, a few years ago, estimated that approximately 20% of the area to which irrigation had been applied in the Western States had been damaged or rendered unfit to produce full crops from water-logging or accumulation of alkali. Other estimates place the damaged area as high as one-third the irrigated area.

The problem of drainage of irrigated lands, therefore, is one which has followed the history of irrigation and which now confronts all engineers interested in this branch of agricultural development. This Committee has endeavored to compile some of the most pertinent facts on the subject in order to place before the Society in concrete form much widely scattered information.

Much research is needed to render more certain the efforts to apply drainage. There are many individuals and organizations engaged in research. If this report encourages other research, it will greatly benefit the profession.

Cause of Necessity for Drainage

Drainage of irrigated lands differs from drainage of humid areas in this particular—under irrigation the control of the source of water is often possible, a condition seldom true under rainfall conditions.

Irrigation is the art of applying water for the growth of crops. If the quantity of water applied is greater than the quantity consumed, the excess must drain away or the land will be water-logged. It is seldom practicable to irrigate so carefully that exactly the quantity needed by the plant and consumed in plant growth is applied to the land. It is not always desirable to add the exact quantity consumed because flushing by excess of water seems essential to health in some plants just as in the case of human beings. Drainage problems arise, therefore, even in the most economically irrigated areas.

Aside from the advisability of removal of plant waste by application of water, it is almost impossible, if maximum crops are grown, to add just the right quantity so that there will be no excess or waste. The variation in soils as to water-holding capacity; the difference in use by crops, and the daily changes in the climatic factors, which influence evaporation and plant transpiration, are all so variable that it is not practical to maintain in the soil the exact quantity of water which may be needed for crop requirements and for the incidental removal of wastes. An excess of water is usually applied—this must be removed by drainage. For the purpose of showing the variability of these factors, the following discussion will be of interest. Canal seepage is the second large factor of the drainage problem and will be discussed subsequently.

WATER-HOLDING CAPACITY OF THE SOIL

The water-holding capacity of the soil must be considered in many drainage problems. One of the most fertile sources of ground-water is that which results from the application of more than the soil will retain. Retentivity of soil for moisture depends on the following:

(*a*) Texture—size of soil grains.
(*b*) Structure—whether puddled or granular.
(*c*) Presence of organic matter, alkali, lime, gypsum or other materials in either the soil or the water.
(*d*) Colloidal matter in the soil.
(*e*) Presence of natural drainage channels or porous material within the soil.
(*f*) Temperature of soil and water.

More specific information about these properties is given later. The results of laboratory investigations on the moisture relations of soils must be used with caution, because no method has been devised of reproducing field conditions in a laboratory sample. In consequence, the results of much laboratory research, while important in consideration of fundamental problems, must be liberally interpreted to fit field conditions.

(*a*).—*Texture.*—Many schemes have been devised of classification of soils by size of particles. That of the Bureau of Soils, U. S. Department of Agriculture, is in most common use in the United States. In this classification the size of soil grains and the relative proportions of each govern the type into which the soil is placed.

Table 1 gives important classes of soils and the mechanical analyses of each.

Other things being equal, the finer the soil grains the greater the water-holding capacity. The relation between texture and water-holding capacity is, however, seldom simple, for other factors enter into the problem to a very large extent.

(*b*).—*Structure.*—The structure of the soil is of great importance in determining either its permeability or its water-holding capacity. Structure defines the arrangement of the soil grains. A soil may have its grains separate—may be puddled as commonly expressed—or may have its particles assembled in granules or aggregates, each of which has somewhat the action of a larger particle. In soils containing fine material, particularly clay, the structure is highly important, as is shown by the difference of the water-transmitting powers of a puddled clay and a friable clay.

Structure, to a great extent, is influenced and controlled by the presence of organic matter, soluble salts in either salt or water, colloidal material, and the previous handling of the soil. There is no definite rule which can be followed in the estimation of these factors; experience alone must guide the investigator. As stated in another place, determination of the water-holding capacity and the permeability of soil in the field is the only certain method of arriving at dependable results.

TABLE 1.—MECHANICAL COMPOSITION OF VARIOUS SOIL CLASSES.*
(U. S. Bureau of Soils' Classification.)

Classification of soils.	Fine gravel, 2–1 mm.	Coarse sand, 1–0.5 mm.	Medium sand, 0.5–0.25 mm.	Fine sand, 0.25–0.10 mm.	Very fine sand, 0.10–0.05 mm.	Silt, 0.05–0.005 mm.	Clay, less than 0.005 mm.
Coarse sands	12	31	19	20	6	7	5
Sands	2	15	23	37	11	7	5
Fine sands	1	4	10	57	17	7	4
Sandy loams	4	13	12	25	13	21	12
Fine sandy loams	1	3	4	32	24	24	12
Loams	2	5	5	15	17	40	16
Silt loams	1	2	1	5	11	65	15
Sandy clays	2	8	8	30	12	13	27
Clay loams	1	4	4	14	13	38	26
Silty clay loams	0	2	1	4	7	61	25
Clays	1	3	2	8	8	36	42

* From U. S. Department of Agriculture, Bureau of Soils, *Bulletin* 78, p. 12, 1911.

(*c*).—*Organic Matter, Lime, Gypsum, and Alkali.*—These constituents greatly affect the water-holding capacity. The first three usually increase the water-holding capacity and at the same time increase the permeability. Alkali usually puddles the soil and renders it difficult to leach. The amount of these salts greatly affects results. For example, small amounts of one salt will increase the water-holding capacity and greater amounts of the same salt will decrease it.

(*d*).—*Colloidal Material.*—Recent research in soil science has impressed upon investigators the importance of colloidal material, not only as influencing the water-holding capacity, but as affecting plant food retention and all activities of the soil and its relation to microscopic organisms and to plant life.

A high percentage of colloidal material has the effect of rendering a soil troublesome when leaching is conducted. The colloidal material absorbs saline material, releases it slowly under leaching, thereby causing a dispersion of the finest grains. The soil then becomes impervious, bakes hard upon drying, and is refractory to agricultural operations.

(*e*).—*Drainage Channels Within Soil.*—Larger openings than the interstices between the soil grains are found in nearly all soils. This is the reason why laboratory studies of movement of soil moisture are difficult to apply to field conditions. Insect burrows, root cavities, cracks, and other larger interstices are found in both soil and subsoil and to great depth. The larger part of the movement of water—downward usually—follows such openings. Capillary movements in the finer cavities and the soil interstices act in all directions. The larger openings, however, have little effect on capillary movements, but control drainage to a great extent.

(*f*).—*Temperature.*—This affects both capillary activities and movements by gravity. In the one case the difference in surface tension of the liquid is affected; in the other, variation in viscosity of the liquid is the measure of the effect.

Crandall* has shown a general relation between temperature and reservoir and canal seepage, which compares well with viscosity changes due to temperatures (Table 2).

TABLE 2.—Effect of Temperature on Surface Tension and Viscosity of Water.

Temperature, in degrees centigrade.	Surface tension, in dynes per centimeter.	Coefficient of viscosity.
0	75.6	1.7921
10	74.2	1.3077
20	72.8	1.0050
30	71.4	0.8007
40	70.0	0.6560
50	68.6	0.5494

The water-holding capacity of a soil under field conditions results from a combination of all these factors. Field conditions cannot be duplicated accurately in the laboratory, consequently it has been found necessary to make determinations of moisture in the field to ascertain the quantity of water which will be absorbed and held. This can be done best by making moisture determinations of the soil before and after irrigation.

Excessive Application of Water as Cause of Necessity for Drainage

Deep percolation from irrigated fields is often a large contribution to ground-water and, therefore, an important source of water to be removed by drainage. This statement is contrary to the opinion of some engineers and many water users, but where detailed study has been made of the subject, it has been found usually that the farmer often contributes through excess application more water than results from canal seepage or any other source. Many factors affect the situation and no such general rule can be applied to all cases without consideration of the local conditions.

The conditions which make for rapid movement of the water in soils also produce small capacity and cause large percolation losses from both canals and irrigated fields. Coarse sand soil or pervious subsoil are potent causes of large movements of water to underground strata. Long lands or checks over which water is spread, and the use of small heads of water in irrigating large checks, both result in deep percolation. Probably the most important cause of all is the habit of farmers to irrigate heavily in order to store up water in the soil on the theory that the number of irrigations can be decreased or that water can be stored in the soil to meet expected deficiency in late summer.

On average soils regularly irrigated, the application of 1 in. of water to each foot of depth to which the soil is to be moistened will fill the soil interstices with as much water as can be absorbed by the soil and retained against the pull of gravity. Thus, 6 in. of water will irrigate the land to a depth of 6 ft. Many soils require less; in sandy lands half this quantity is all that is necessary. When the soil is very dry, more than 1 in. is required, for example, the first irrigation of annual crops. When the soil already contains moisture, as it does where perennial crops are grown, the amount of moisture present prior to irrigation should be considered in determining the quantity necessary for each irrigation.

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* *Engineering News-Record,* February 13, 1919.

The use of much larger quantities than those indicated is the rule in many irrigated areas. Testimony given by farmers in a recent irrigation water suit indicated the average use of 12 in. or more in depth per irrigation in soil which would hold that quantity. Rules for the distribution of water in some irrigation districts permit the use of 9 to 12 in. for each irrigation on soils which should be thoroughly irrigated with one-half to one-third that quantity of water.

If scientific preparation of the land and the application of the quantity of water which is absorbed without waste could be insisted upon, much of the necessity for drainage could be avoided. This condition is seldom practicable; drainage, therefore, becomes a necessary part of most irrigation projects.

TABLE 3.—MOISTURE RELATIONS OF IRRIGATED SOILS.

Soil type.	Wilting point*, percentage. (1)	When irrigation is desirable†, percentage. (2)	Maximum field capacity,‡ percentage. (3)	Weight per cubic foot§. (4)	INCHES RETAINED‖ PER FOOT OF SOIL.	
					Columns (1)-(2). (5)	Columns (2)-(3). (6)
Sand	3	5	8	100	0.38	0.58
Sandy loam	5	9	13	95	0.73	0.73
Fine sandy loam	6	12	18	90	1.04	1.04
Loam	8	14	21	88	1.01	1.18
Silt loam	10	16	23	85	0.97	1.14
Light clay loam	13	17	24	82	0.63	1.10
Clay loam	14	18	24.5	80	0.61	1.00
Heavy clay loam	16	19	25	78	0.45	0.90
Clay	18	20	25.5	75	0.29	0.79

* The wilting point is the percentage of moisture at which crop plants commence to wilt and below which moisture should not be allowed to fall.

† This is the percentage at which irrigation is ordinarily given. It is considerably above the wilting point, particularly in the soils of medium texture. Soils, with reasonable care, may be allowed to approach nearer to the wilting point in dryness than is shown in Table 3.

‡ Maximum field capacity represents the percentage of moisture which a long column (5 ft. or more) of the soil will retain a few days (generally 3 days) after it has been irrigated. It is the practical maximum water-holding capacity of the soil.

§ This is the weight per cubic foot of dry soil. This weight varies considerably.

‖ Columns (5) and (6) give the calculated depth of water required to raise the moisture percentage as shown,

$$\frac{\text{weight per cubic foot} \times \text{percentage increase in moisture} \times 12}{62.5}$$

To illustrate the moisture-holding capacity of various types of soils, Table 3 shows the average percentage of moisture at the wilting point, the point where irrigation water is ordinarily applied, and the maximum field capacity of the soil. These figures are given for purposes of illustration only and should not be taken as exact for any particular locality or group of soils, because the weight of soil per cubic foot varies and small changes in amounts of organic matter, alkali, or colloidal material, may produce wide variations from these averages. In any region where such problems are studied, determinations of the moisture relations of the soils should be made, in order to furnish the basis for more accurate estimate of the water-holding capacity of the lands being irrigated.

The irrigation of soil to a depth of 5 ft. is all that is ordinarily required. The deep percolation which, to some extent at least, cannot be prevented, will moisten the soil to a greater depth. Crop roots in irrigated areas seldom penetrate, to any extent, beyond 6 ft. The zone of most important root growth is from 2 to 5 ft. If enough moisture is applied to moisten the 5 ft., with a reasonable allowance for deep percolation, the irrigator can be assured that the full root zone will be irrigated.

A compilation giving acre-inches required to irrigate soil to a depth of 5 ft., with 25% for waste and deep percolation, is shown in Table 4.

TABLE 4.

Soil type.	Inches required to moisten 5 ft. of soil without waste.	Inches required to moisten 5 ft. of soil, with allowance of 25% for waste.
Sand	2.90	3.62
Sandy loam	3.65	4.56
Fine sandy loam	5.20	6.50
Loam	5.90	7.38
Silt loam	5.70	7.13
Light clay loam	5.50	6.88
Clay loam	5.00	6.25
Heavy clay loam	4.50	5.63
Clay	3.95	4.94

If the figures given in Table 4 are compared with the quantities of water per irrigation in any irrigation area, it will be seen that in few of them is the quantity of water used per irrigation as small as is shown in the column in which an allowance of 25% is made for waste. The excess water goes to ground-water and causes it to rise.

Canal Seepage as Contributing to Drainage Problem

Seepage from canals contributes directly to the drainage problem. In some cases it may be the most important source of rising ground-water. In all drainage investigations, seepage from canals has some bearing on the problem and should be investigated as an element in the drainage study.

Canal seepage or percolation may be expressed in several ways; ordinarily, it is given as a percentage of the water carried. For purposes of comparison of canals and materials composing canal beds and banks, it should be expressed in depth per unit of bed area, that is, percolation per day in cubic feet per square foot of wetted area.

Canal losses consist of both seepage and evaporation. The latter is usually neglected because it amounts to less than the error in measuring the total losses. As an example, take the case of a 1 000-sec-ft. canal with 100 ft. of water surface, where the total losses amount to 1% of the flow. This equals 20 acre-ft. per day. The water surface covers 12 acres. The total losses are 1.67 acre-ft. per acre of water area per day. Evaporation seldom exceeds ½ in., or 0.04 ft., in depth per day, or 2.4% of the total losses. Since this is less than the probable error in measurement of water, it is ordinarily neglected.

No exact relation between character of bed material and rate of percolation can be given. Generally, it may be said that coarse material has a high rate of percolation and fine material a low rate. A clay with joints and cracks well developed may have a high rate and, again, a gravel with the interstices well filled with fine sand may be very tight. Measurements alone can establish the facts. As examples of losses in canal systems and of losses in various materials, Tables 5 and 6 are quoted.

TABLE 5.—Seepage Losses.*

Project.	Total Losses, Percentage.								Predominating soils.
	1912.	1913.	1914.	1915.	1916.	1917.	1918.	Mean.	
Salt River	40.0	40.0	42.7	40.0	43.3	43.7	45.9	42.2	Sandy loam and clay.
Yuma	34.4	32.0	27.7	20.0	27.4	25.3	23.2	27.1	Rich alluvium.
Orland	20.0	26.5	26.0	27.3	20.5	27.3	33.1	25.8	Sandy loam, silt, and gravelly loam.
Uncompahgre	4.8	5.1	7.6	12.4	2.2	2.9	7.7	6.1	Red sandy gravel, adobe, and clay loam.
Boise	45.5	33.9	34.8	37.1	34.8	35.8	35.1	36.7	Clay loam and light sandy loam.
Minidoka:									
North Side Gravity	26.0	22.6	25.0	36.3	43.9	48.9	41.4	35.2	Sandy loam, clay loam, and volcanic ash.
South Side Pumping	35.1	32.9	35.1	36.3	38.8	36.6	35.6	35.8	Sandy loam, clay loam, and volcanic ash.
Huntley	16.2	21.6	28.6	41.3	40.3	48.9	47.5	34.9	Clay and sandy loam.
Milk River	42.8	32.7	40.8	61.4	38.5	41.4	35.3	41.8	Sandy loam, clay, and gumbo.
Sun River	20.0	38.1	39.7	41.8	41.9	37.6	40.3	37.1	Sandy loam, clay, adobe, and alluvium.
Lower Yellowstone	43.0	52.0	52.4	43.1	47.9	42.8	46.9	46.9	Sandy loam and gumbo.
North Platte	38.7	38.4	36.8	66.0	40.7	47.2	37.7	43.6	Sandy loam.
Newlands	39.8	36.7	29.9	41.2	29.9	32.2	38.0	35.4	Sandy loam, clay, and volcanic ash.
Carlsbad	48.0	54.5	55.5	48.6	58.5	57.2	50.1	53.2	Pecos sandy loam, large lime content.
Rio Grande	....	6.8	2.7	5.1	6.5	3.7	14.9	6.6	Rich alluvium.
Umatilla	27.8	27.4	40.1	43.8	36.5	24.8	32.8	33.2	Sandy loam.
Klamath	36.1	39.8	40.1	50.3	49.1	41.5	40.4	42.5	Disintegrated basalt, volcanic ash.
Belle Fourche	31.7	40.2	43.6	31.1	15.7	31.5	32.6	32.3	Clay and sandy loam.
Okanogan	47.1	38.4	21.1	24.1	22.2	20.4	33.7	29.6	Volcanic ash, sand, and gravel.
Yakima, Sunnyside	26.7	25.8	28.4	31.2	26.2	26.8	23.4	26.9	Sandy loam and volcanic ash.
Yakima, Tieton	16.6	21.4	27.2	27.4	27.3	26.2	27.8	24.8	Volcanic ash.
Shoshone	36.4	36.5	37.2	35.1	34.9	41.8	37.7	37.1	Light sandy loam and clay loam.

* *Reclamation Record*, April, 1921, p. 181, Table 1.

Frequently, water-logging is caused by excessive seepage from certain parts of canals. Where a detailed study is made of canal losses, such sections of high percolation may be located. The lining of a few selected localities may greatly decrease the seepage menace. Effective removal of excessive water has resulted from lining or puddling of short sections of leaky canals.

Movement of Ground-Water

Once in the subsoil, the ground-water originating in canal percolation as the result of excessive application by water users flows under the force of gravity down the slopes of the underground strata. Such movements are uncertain as only very meager information as to the character, extent, or direction of the underground strata can ever be obtained. Inability to obtain such information makes the prediction of the rate or direction of movement of ground-water uncertain, and, for that reason, it is essential in irrigated areas to collect information with great regularity, from which these movements of ground-water can be deduced.

For the purpose of determining the movements of ground-water, observations on water levels are of great assistance. These records should be kept, if possible, from the time water is first turned into the irrigation canals. Since water may collect and move at different levels, flowing along impervious materials, the observations should include both shallow and deep wells. When equilibrium has been established, or where it has been demonstrated that there is free movement between deep and shallow strata, surface wells alone will be all that is required.

In many irrigated areas, records have been kept on water levels over long periods of time. In the Salt River Valley of Arizona these records cover a period of more than 10 years. In the Modesto and Turlock Districts, wells approximately at every section have been read at intervals over a period of 13 years. These wells are located so as to be as free as possible from local influences of ditches or pumping plants and give, by comparison with other years, a close approximation of the fluctuations of the water surface.

TABLE 6.—LOSSES PER DAY IN CANALS OF THE U. S. RECLAMATION SERVICE.*

(Depths, in feet per square foot of wetted area.)

Soil.	Number of observations.	Maximum.	Minimum.	Mean.
Gravel and sand	2	3.62	1.44	2.53
Gravel	38	7.05	0.22	1.70
Gravel and rock meal	9	2.84	0.88	1.56
Sand	5	1.79	0.81	1.27
Rock meal	18	1.73	0.38	1.24
Sand and loose rock	7	0.86	0.62	0.72
Sandy loam	60	3.70	0.01	1.01
Loam	5	1.44	0.84	1.07
Sand and volcanic ash	21	1.66	0.22	0.90
Gravel and volcanic ash	5	1.54	0.38	0.88
Volcanic ash	8	1.16	0.56	0.82
Brule clay	4	1.11	0.72	0.91
Clay and gravel	18	1.34	0.16	0.79
Clay and sand	12	1.43	0.50	0.76
Adobe	9	0.93	0.46	0.65
Hardpan and loose rock	2	1.12	0.24	0.68
Clay and shale	7	1.02	0.30	0.59
Volcanic ash, clay, and hardpan	4	0.77	0.46	0.60
Cemented gravel and sandy loam	2	0.47	0.40	0.44
Caliche	2	0.49	0.38	0.43
Clay	12	0.77	0.11	0.34
Clay loam	12	1.12	0.06	0.30
Gumbo and sandy loam	11	0.77	0.12	0.29
Concrete lining	9	1.07	0.06	0.33
All soils	277	1.66	0.43	0.87

* *Reclamation Record,* April, 1921, p. 182, Table 2.

In the Merced Irrigation District, both deep and shallow wells are observed. The shallow wells are located at frequent but irregular intervals throughout the district. Some of these wells are used to determine the general ground-water conditions, and they are established as far as possible from local influences, such as canals, drainage wells, or other sources of fluctuating ground-water. Other wells are located and read to collect information on local conditions, such as water levels around drainage wells or in localities threatened with high ground-water. These data are promptly plotted and made available for use. Engineers in operation work, land owners, realtors, bank appraisers, and others use such records a great deal.

EFFECTS OF LACK OF DRAINAGE ON IRRIGATED LANDS

The effects of lack of drainage on irrigated lands are shown in several ways. In part, the effects are only temporary, but in some respects they are more or less permanent.

1.—Water-Logging.—This is usually the first noticeable effect. This term is one in common use among irrigators and means an excess of water, over that essential to plant growth, for a sufficient length of time that the normal functions of roots are affected. Most agricultural plants require both air and water around the roots. When water is present in such quantities that circulation is retarded and air is not present, the roots cannot function normally.

The effects of water-logging are to retard root growth and to check plant development. This effect may be important, even though the plant continues to live and have a healthy appearance. Partial water-logging is manifested by reduction in yield and may not be apparent to the eye. The relations between water applied and plant yield have been determined in many places. Typical of these are the results of five years' (1910-14) irrigation of alfalfa at the Davis Farm by the University of California. The results appear in Table 7. It is apparent that beyond 30 in. of water, there is no practical good accomplished. Applications in excess of 48 in. cause lower yield.

TABLE 7.—RELATION BETWEEN AMOUNT OF IRRIGATION AND YIELD OF ALFALFA, DAVIS, CALIFORNIA.

Irrigation, in inches.	Hay, in tons.
0	4.42
12	5.90
18	7.04
24	7.81
30	9.12
36	9.40
48	9.43
60	9.19

2.—Rise of Alkali.—In the sense used by irrigators "alkali" means water-soluble salts. Any water-soluble material falls in this classification. While the number of water-soluble salts found in soils is large, practically only a few are of sufficient prominence to have any effect. Investigations have shown that in some localities small quantities of rare salts have highly destructive effects, as, for example, borax or barium salts. The broad classification into "black alkali" and "white alkali" has long been accepted and is yet commonly used, although it is now known that this general classification may not always be applied.

Black alkali includes salts alkaline in the chemical sense. Sodium carbonate and sodium hydrate are the important salts. Such salts have, through their caustic nature, the power to dissolve organic matter, the resultant solution when concentrated appearing black, thus accounting for the popular name.

The so-called black alkali salts are white when pure and many times in desert soils the color is no indication of the kind of alkali present. The corrosive action of the soil on vegetable matter renders such salts very toxic, and amounts as low as 0.10% will usually prevent plant growth, while one-half or one-fourth this quantity may render agriculture unprofitable through the effect on the permeability of the soil. The alkaline nature re-acts on the mineral grains, dissolving silica and puddling the soil grains.

Such soils bake hard when dry and pass water slowly. Leaching does not remove the salt promptly. The gelatinous surface of the soil grains caused by the corrosive action on the silicates and quartz grains retains the soluble material almost indefinitely. Minute quantities of the salt cause the soil to be dispersed and makes leaching difficult. Chemical neutralization of the salt alone seems to correct the unfavorable soil characteristics. Gypsum (calcium sulfate) and sulfur have been found to change the black to white alkali. The cost of these chemicals is so high that it is not practical to carry out reclamation of black alkali salts except when small amounts only are present. Leaching to remove all but a small quantity and chemical neutralization of this last trace seem to be the most practical methods of reclaiming such lands. The time required for such action is long and it can be accomplished only where drainage is easy, the water cheap, and gypsum available at a low price. In spite of many attempts there has been no practical reclamation of any extensive areas of black alkali land.

White alkali salts are principally chlorides and sulfates of sodium, potassium, and magnesium. They occur mixed in all proportions. They have comparatively little effect on the physical properties of the soil and may be leached out much more readily than is the case of black alkali salts. Such salts are less toxic to plants. The limit of tolerance varies with the plant, the kind of soil, and other factors, but ordinarily 0.005% of white alkali salts in the soil is the limit in land which can be farmed. One-half this amount frequently causes serious losses.

White alkali salts are not retained by the soil as persistently as black alkali. For this reason they may be washed out of land by flooding and leaching. Large areas of land have been washed free of salt and alkali and profitably farmed.

3.—Changes in Physical Condition of the Soil.—Changes in the physical condition of the soil due to lack of drainage are of great importance. Land becomes harsh, difficult to cultivate, cloddy, and generally refractory. These changes frequently are noticed before there is any large accumulation of alkali and they persist after the drainage has become established and the salts leached out.

The re-establishment of good tilth in drained soil is the most difficult part of the reclamation process. Lands have frequently been drained and the alkali removed by leaching, only to find that the soil was so refractory that crops could not be grown. A long period was then required to incorporate organic matter, and restore in the soil the quantities which cause the land to be mellow and cultivable. This process is slow, frequently requiring years, and knowledge of how best to restore good physical condition in the soil is limited. An opportunity for research into this question still awaits the Agricultural Department and Experiment Stations.

4.—Losses in Plant Food.—Excessive use of water results in the leaching out of plant food from the soils. Comparatively little is known as to the extent to which this goes on, but it is probably an important factor in the decreased yield in many irrigated areas.

PREVENTION OF THE NEED OF DRAINAGE

The drainage problem can be at least partly solved in many cases by the removal of the cause of excess water in the soil. Three methods have proven of benefit:

1.—Control of the farmer's use of water.
2.—Prevention of canal seepage.
3.—Pumping from irrigated areas.

These methods are discussed in more detail as follows.

1.—Control of the Farmer's Use of Water.—This is in effect an attempt to control the use of water so that there is added to the soil at any time only the quantity the soil can absorb and retain against the force of gravity. The period between irrigations then being under control, the deep percolation losses may be reduced to a minimum.

In application of this method, the first and most important matter is the determination of the water-holding capacity of the soil. The use of laboratory studies of soil types and the determinations of moisture equivalent and water-holding capacity by laboratory methods are some help in forming approximations, but actual field determination of water-holding capacity is a more dependable method. Laboratory studies of soils do not show accurately field conditions, because it is never possible to place laboratory soils in the same physical conditions as are found in the field.

With an accurate knowledge of the absorptive capacity of each type of soil, the attempt should be made to direct the preparation of the land in such width and length of runs that this quantity of water can be absorbed with the use of a head of water of practical size.

With lands prepared properly, the size of the head (or quantity of water used at one time) determined, and the period of application found by experiment, the question of the frequency of irrigation is then to be determined. This, of course, is dependent on the weather, the time of year the crop is growing on the land, and the amount of moisture remaining in the soil.

The practical application of this method of water control is thus seen to be a complicated matter, into which many factors enter. In addition to the physical factors of soil, climate, and crop, the human factor of how to encourage the farmer to apply these principles enters seriously into the problem. Such things become a matter of education of the water user. The drainage engineer is concerned with the education of the water user in better practices in the method and rate of application, but practical difficulties are met in any attempt to enforce correct use of water.

Rules for the distribution of water are a help, but their enforcement is often difficult of accomplishment. Rules usually require the farmer to keep his ditches in condition to handle water and to have his land prepared so that the water can be economically handled. How seldom is the strict enforcement of such rules carried out? One of the most practical rules is that in use by many irrigation districts in which a time limit is placed on the use of water; as examples, the Modesto Irrigation District delivers a uniform head of 25 cu. ft. per sec. and limits the time of use to 40 min. per acre of land; the Turlock and other districts deliver 15 sec-ft. and limit the time to 30 min. per acre. Of similar effect is another type of rule, which prohibits the use of more than a given quantity per acre in one irrigation. The Corcoran Irrigation District's rules prohibit the use of more than 6 in. in depth in any one irrigation.

Placing a penalty on excessive use by a high charge for water in excess of a minimum quantity has long been applied to restrict use. This method has been only partly successful, although it has been of some benefit. While it may not altogether prevent waste, it has the satisfying effect of placing at least some of the burden of the cost of drainage on the lavish user of water.

The control of the use of water so that there will be no waste is therefore, theoretically, a method of promise. Practically but little can be accomplished in a short time and the engineer is forced to attack the problem from the angle of removing the excess of water. Discouragement should not prevent the continued attempt to reduce the losses into deep soil by rational practices of irrigation. Continued effort has produced results and more will no doubt follow. More immediate results will follow other lines of attack.

Excessive care in the preparation of land and in the application of water costs money. The farmer frequently finds it cheaper to pay for drainage than to use care in the handling of the water. This is particularly true in irrigated districts, where the land is newly prepared. The first attempt to fit the land for the distribution of water does not always result in economic distribution. Sometimes the land settles and becomes irregular; at other times, lack of capital or inexperience are the causes of the trouble.

It becomes practically impossible in many such cases for the practicing engineer to enforce rules requiring the land to be well prepared, and, consequently, he is forced to get along the best way possible. The result is wasteful use of water and a drainage problem.

2.—Prevention of Canal Seepage.—Losses by seepage from canals account for from 25 to 50% of the water diverted from rivers. These losses can be almost entirely prevented by lining canals with impervious materials. The cost of lining is high, and it becomes a problem in economics to determine whether some other method is less costly. For example, it has been estimated that the lining of all canals in the Salt River Valley of Arizona will cost $8 700 000. To do this as a prevention of drainage would save the removal by drainage of 170 000 acre-ft. of water. The annual cost of interest, depreciation, and repairs, all taken at 8%, would be $700 000. This same quantity of water could be pumped from a depth of 30 ft. at a cost of about $1.50 per acre-ft., or a total of $250 000.

Since the source of water causing the need of drainage originates only in part in seepage from ditches, the lining of canals will not altogether remove the necessity for drainage. Usually the lining is carried out in the sections of ditches where canal losses are heaviest.

The result is that only portions of canals are lined. In well-established districts which have been under operation for some years, it is the policy to provide a sum of money each year for continuation of ditch lining. Thus, in the Turlock Irrigation District, about $50 000 is provided in each year's budget for new lining. In the Merced and South San Joaquin Irrigation Districts from 5 to 10 miles of canal are lined each year. Few irrigation projects can afford to provide all ditches with lining at the time of the construction of the system, particularly projects where interest must be paid on funds spent on construction work. The result is the budget method of providing annually a sum to be devoted to concrete lining.

Accurate records of losses from canals by seepage are seldom available. Since many drainage problems are a determination of the extent of canal seepage as a contributing cause to the necessity for drainage, it becomes necessary in every case to determine the relative extent of seepage from various materials in which canals are built.

The U. S. Reclamation Service published in 1921 the results of seepage measurements in many kinds of soils.* The seepage varies greatly with local conditions. The presence of alkali in the soil, thickness of strata, and particularly the presence of sand, gravel, or other coarse-grained material, all greatly affect the result. Accurate results are obtainable only after careful measurements.

For purposes of rough estimation of canal losses, the figures in Table 8 may be used.

3.—Pumping from Irrigated Areas.—Pumping of ground-water from irrigated areas lowers the water level and produces the same effects as drainage by any other means. In many regions this drainage effect has been produced as a by-product of the effort to secure ground-water for irrigation. Notable examples of this are in the irrigated areas on the Kaweah and Kings Rivers in the San Joaquin Valley of California.

Thirty years ago in the San Joaquin Valley, large areas of land were out of cultivation or producing meager crops because of the rise of ground-water and alkali following irrigation. Some of the colonies near Fresno, originally considered as the most fertile in the neighborhood, were to a large extent abandoned because of rise of water from 60 ft. or more to a few feet below the surface. The situation was nearly as bad under some of the canals diverting water from the Kaweah. The University of California established an experiment farm on the Kaweah Delta, near Tulare, to study alkali. At this farm some of the classical experiments on alkali lands were conducted by Dr. Eugene Hilgard. Both the areas mentioned were watered by canals diverting from unregulated streams. Water was frequently deficient after the middle of summer.

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* *Reclamation Record*, April, 1921.

TABLE 8.—APPROXIMATE LOSSES BY SEEPAGE CANALS IN VARIOUS MATERIALS.
(In feet in depth per square foot wetted.)

Soil material.	Area per day.
Gravels and coarse sand	1.5 to 2.0, or more.
Sand soil	1.05 " 1.5
Sandy loam	1.00 " 1.25
Loams	0.60 " 1.00
Clay loams	0.40 " 0.60
Clays	0.25 " 0.40
Concrete lining	0.25, or less.

The practice prevailed of heavy irrigation in early summer in an attempt to store water in the soil for late summer use by crops. This resulted in high ground-water. The improvements in gasoline engines which made them effective for farm use, about 1900, encouraged pumping as a supplemental supply. Electric power on farms became available to many irrigators after about 1908. Pumping became general, both as supplementary to canal water and as the supply for many farms having no canal rights. Around Tulare nearly all the canal water is pumped. The result is that the ground-water plane has lowered, lands once menaced by high water have been relieved of that menace, and alkali areas have begun to improve. Drainage has thus been effected as an incident to recovering ground-water.

The fact that pumping as a source of part of the water supply will relieve the drainage menace has become recognized practice among some engineers. The Merced Irrigation District of California in its original plans provided for securing 80% of its supply from the river by direct diversion and storage and 20% from ground-water. Similar provision has been made by the Kern River Conservation District, the San Joaquin Water Storage Districts, and other proposed irrigation projects.

This theory has been put into practice by other operating concerns. In the Salt River Valley, of Arizona, pumps capable of delivering nearly 500 000 acre-ft. annually from ground-water have been installed. The Salt River Project now depends to a certain extent on ground-water to carry it through dry cycles. Here, water is drawn far below the depth required to effect drainage at certain periods. The result has been effective recovery of water-logged and alkali areas.

DRAINAGE INVESTIGATIONS

The investigation of drainage problems involves the collection and interpretation of data of several varieties. Like many other engineering investigations, records over a period of time are of importance. It is difficult to arrive at dependable conclusions with observations made over a short time.

The prediction of the necessity for drainage in any newly irrigated region is difficult, because a great deal depends on the irrigation practices of the water users, and the movements of ground-water are a matter of conjecture. With so many unknown factors, prediction of either amount of drainage or place where high ground-water will develop, is very uncertain. For this reason it is highly important that records be collected and preserved at all times of the height of ground-water beneath irrigated areas, of the quantities of water run through canals, and of the seepage losses from canals and laterals. It is further desirable to have collected facts on the rate and extent of the use of water by irrigators.

If such information is collected from the time water is first applied to the land, the history will be sufficiently complete for the drainage engineer to design a method of taking care of the excess water. Lacking such information, it is necessary to delay drainage relief until a full year's records are collected. With periodical analysis of such records, it is possible to predict with some degree of certainty the time when drainage relief must be applied.

It has now become established practice in many well-organized irrigation projects to collect with great regularity the depth to ground-water beneath irrigated areas and to watch carefully such movements as occur. By such means the ground-water level can be controlled and held at the most desirable level. As examples of such collection and use of ground-water records, the following may be mentioned as typical. In the Salt River Valley of Arizona, records of levels of water in wells are collected at regular intervals throughout the year. The wells observed are in part auger holes and in part domestic or other wells. These records are promptly posted on maps. Land owners, bank appraisers, real estate salesmen, and many others have frequent use for these records. The project officials watch changes in water level with great interest.

In some California irrigation districts, water levels are observed with regularity. In the Turlock District, about 190 wells are measured both summer and winter. These are nearly all auger holes located at section corners or close by in some place as free as possible from local influences. In the Modesto District, 75 section corner wells are observed monthly. In the Merced Irrigation District, one man is employed reading water levels in wells. This district is relatively new and a reservoir was placed in operation in 1926. Ground-water conditions have changed rapidly and prompt information as to changes in water levels was essential.

For observation of ground-water level and determination of rise and fall with respect to the land surface, it is not necessary to connect such wells with levels so that they may be plotted with reference to a common datum. Levels are advisable, however, for the determination of the ground-water slope, both in direction and degree of slope, which are important when drainage is being studied. Maps showing depth of water below ground level and ground-water contours are both important.

ALKALI AS ASSOCIATED WITH DRAINAGE

The term, "alkali", is generally used, in the irrigated regions, to mean any soluble mineral salts in soils. These salts result from the decomposition of rocks and minerals and remain in the soils because there is not sufficient water passing through the soil to leach them out, or they are absorbed by the surface of the soil grains. The salts present are largely chlorides, sulfates, carbonates, and bi-carbonates of sodium potassium, calcium, and magnesium. Occasionally, other salts appear; for example, nitrates are reported in some soil analyses.

Two reasons can be advanced to account for the presence of soluble salts in irrigated soils: First, the amount of rainfall is insufficient to leach out and wash away the soluble products of soil and rock decomposition; and, second, evaporation is great owing to the dry atmosphere, and where the ground-water is so close that capillary forces can draw the water to the surface, there is the constant tendency for an increase in soluble salts near the surface. Movement of salts by capillarity is more rapid than by water moving downward by gravity, because in gravity movement the larger pore spaces only are leached; the fine capillary spaces have little movement. In capillary movement, water moves through the finest spaces.

The drainage engineer is interested in alkali in several ways. The first and most important point is that where alkali is present it is necessary to keep ground-water levels so low that capillary forces cannot raise water to the surface. There, too, the movement of water must be downward, that is, the quantity of water used must be so great that there is drainage downward, otherwise soluble salts will increase near the soil surface.

Alkali salts, if present in considerable quantity, may have the effect of deflocculating the fine soil particles, causing the soil to be puddled. Leaching, when the soil is puddled, is slow and difficult. The difference in rapidity with which soils may be leached is very great. In some cases the leaching process proceeds rapidly; in others, it is so slow as to be impractical in operation. The reason for this difference is not known in all cases, but it may be generally observed that soils rich in lime and organic matter may be readily leached, whereas soils poor in these constituents are difficult to handle.

For practical purposes, alkali has long been classified into black alkali and white alkali.

Black alkali, carbonate of soda, has an alkaline reaction in the chemical sense and is corrosive of vegetable matter, and much more destructive than the white alkali salts. It further has a puddling effect on the soil, renders it difficult to leach, and causes it to bake and become refractory when irrigated. Black alkali cannot be easily leached from the soil. No practical method of draining and reclaiming such lands has been devised. Black alkali can be neutralized by chemicals, such as sulfur or gypsum (calcium sulfate), but these processes are so costly that they can be used only on small areas, where the value is high or where the quantity of salt to be neutralized is very small.

White alkali salts, in the classification of the textbooks, include the remaining soluble salts in the soil. These are neutral in the chemical sense. The statement has long been accepted that the removal of white alkali salts by water and drainage is a simple matter and easy of accomplishment. Whether or not this will prove true depends on the composition of the soil.

Recent research has made great changes in the knowledge of alkali in soils and the old statements must be modified to some extent. Alkali can no longer be classified into black and white alkali, but many other factors should be taken into consideration.

The present conception of the alkali problem is that sodium salts are quite as injurious to the physical condition of the soil when they occur in high concentrations as sodium sulfate and sodium chloride, as when they occur as sodium carbonate. When the leaching of a soil containing sodium salts in excess is undertaken in the field, it is likely to be successful only when there is present a considerable amount of calcium and magnesium. These being absent, the soil becomes impermeable when moist and bakes hard when dry, producing the result formerly attributed to "black alkali".

The problem of alkali land reclamation, when this refractory condition is reached, becomes very difficult. There is, as far as is now known, no certain and practical method of restoring tilth to soil which shows the refractory state due to this alkaline reaction, except that of chemically changing the condition by the addition of gypsum or calcium sulfate. Whether or not this is practical, is a question of cost. Following the removal of the soluble material, the soil must be brought into such physical condition that it becomes acceptable to plant roots and workable by ordinary farm machinery.

The drainage engineer, therefore, is interested in the reclamation of alkali lands and the practical methods of bringing such lands into cultivation after the excess of water and alkali are removed.

Use of Saline Waters in Irrigation

Many Western waters carry appreciable amounts of soluble salts. As the quantity of water diverted for irrigation has increased and the percentage of return water has become higher, the quality of many sources of irrigation water has been changed greatly by the increase of soluble salts in the return and drainage water.

Drainage water, whether recovered by ditches or wells, is frequently saline and its use in irrigation is sometimes questionable. The release of drainage water at times serves to raise the salt content of streams below the drainage outlet. The rights of lower appropriators may thereby be affected by changing the quality of water in the stream.

The use of saline waters in irrigation is a subject beyond the scope of this report, for it is a matter of great complicity, involving the chemical and physical characteristics of the soil and the abundance and proportions of the salts present in the water. A great many facts as to the result of long, continued use of saline waters are unknown. There is here an interesting field for research. The drainage engineer cannot solve all the problems, but a general knowledge of the subject will assist in the consideration of questions which arise where alkali land or saline water are encountered.

In the most general terms, neutral waters (neither chemically alkaline nor acid) carrying more than 1 000 parts of solids per million should receive thorough examination before their use in irrigation is recommended. Below that amount there are records of long, continued use on a great variety of soils without serious damage to the soil, and there is little reason to anticipate trouble in any well-drained soil.

When the total salts exceed 1 000 parts per million, analyses are advisable to determine the proportion of soluble salts which do not crystallize on concentration and which may accumulate in the root zone. There are many cases on record where waters exceeding this limit of 1 000 parts have been used with no visible decrease in fertility of the soil, but there are also places where such waters have proved harmful.

As examples of use of water of high salinity, the following may be mentioned.

Colorado River.—The average of samples from each irrigation at the Experiment Farm of the U. S. Department of Agriculture at Bard, near Yuma, Ariz., shows the total solids in solution for the year's average to vary between 805 and 997 parts per million (October, 1922, to September, 1927). Weighted by flow—as would occur when a stream is completely reservoired—the average yearly salinity will vary from 544 to 697 parts per million. For considerable periods, during low flow, the salinity has exceeded 1 000 parts, reaching in excess of 2 400 parts in October, 1924. Water of this average quality (except for slight changes due to alkali absorbed from the channel below Yuma) has been used in the Imperial Valley for 25 years without damage to well-drained land. Lands subject to seepage or high ground-water have been seriously damaged.

Salt River Valley.—The average salinity of the Salt River Project from Roosevelt Reservoir is high, chlorides predominating. The water from Salt River diverted at the joint head (in part, return seepage) averages 1 678 parts per million.

Several developments of ground-water in and adjoining the area irrigated from Salt River have produced water of salinity in excess of 1 000 parts per million. The Roosevelt Irrigation District Project, now under construction (1927), is designed to use drainage pump water from a part of the Salt River Project with an average salinity of 1 590 parts per million.

Waters of the quality described have been used in the Salt River Valley for more than twenty-five years.

Pecos Valley, New Mexico and Texas.—Waters of high salinity have been used for many years—occasionally exceeding 5 000 parts per million. Good drainage with excessive use has enabled good crops to be grown. The average salinity of three stations on the Pecos River, in New Mexico, is 2 350 parts per million. Lands using such water become badly alkaline where waterlogged. Drainage works have permitted lands to be reclaimed and farmed profitably.

Water supply resources have reached such a point that drainage water frequently has a value which offsets in part the cost of drainage. The quality of the water, therefore, has to be considered by the drainage engineer. As a conspicuous example, the Salt River Project originally started construction of wells for drainage. The water is so valuable that to-day the well development exceeds that necessary for drainage, and in the effort to secure more irrigation water in dry seasons, water is now drawn down lower than needed to drain the land.

In many parts of the San Joaquin Valley, water can be obtained from wells, in areas needing drainage, cheaper than it can be stored in reservoirs in the mountains or secured from any other source. Drainage in such cases is a by-product of the effort to secure water.

In determining the value or suitability of water for irrigation purposes, chemical analyses should be made. In analyzing water it is important to determine the total solids in solution, the amount of lime, magnesium, and of each of the common acid radicals, chlorine, sulfates, bi-carbonates, and carbonates. Sodium and potassium should be determined in enough samples to estimate their approximate amount.

Lime and magnesium salts are of particular importance. Hard waters are desirable in agriculture; the lime in the water exchanges with sodium in the soil, the result improving the soil. As has been expressed, "hard waters make soft land." On the other hand, soft waters, particularly those alkaline in reaction (commonly called black alkali waters), may have the opposite reaction and in time change soft friable soil to hard land of refractory nature.

Neither of these changes takes place in the soil rapidly. Time—sometimes several years—is required before the full effect of the water is apparent. Much depends on the soil. Lime in abundance in the soil retards the effect of soft waters. A light soil easily permeated by water is not affected as is a heavy soil or one with a large amount of colloidal material.

The most important consideration in the use of saline waters is to have free and rapid drainage so that the land does not become water-logged. Care should then be taken that enough water be used to cause downward flushing into the drainage channels.

Reclamation of Land After Application of Drainage

The most difficult part of many drainage projects in irrigated areas is the improvement or rehabilitation of the land, if such a word may be used, after drainage has removed the excess of water. This feature of the work is a part of the drainage engineer's activities, at least he should know enough about the matter to decide whether reclamation is possible or that there is a reasonable assurance of its success.

Land water-logged without accumulation of alkali is usually in physical condition for agriculture, as soon as the water is removed. Frequently the land is more fertile than before it was water-logged, because of the large amount of vegetable matter which remains in the earth from water-loving vegetation. Reclaimed swamps are usually of high fertility. Acid soil may be troublesome, but methods of neutralization of acid soil have been devised and, besides, many crops require acid soil for successful production. Acid soil is unusual in the arid regions, because of the abundance of lime.

Where alkali has accumulated, the improvement of the soil after drainage may be difficult. The clay in the soil may be dispersed, the soil rendered impervious, leaching difficult, and the land harsh with a tendency to bake and form clods. The organic matter may have been dissolved and washed away. In such cases, there is a tendency for the soil to become alkaline (in the chemical sense), or give those characteristics attributed to "black alkali". This condition may arise in the leaching of land which does not show black alkali characteristics at the time leaching commences—the alkalinity develops in the process of washing the soil.

The theories of agricultural chemists regarding alkali in soils have changed a great deal since 1917. Much important research is now going on in a number of States, in the U. S. Department of Agriculture, and in foreign countries. A drainage engineer will hardly find it possible to keep in touch with the latest information published by these research scientists. About the best he can do is to refer his alkali problems to some agricultural chemist familiar with most recent research.

The belief, prevalent among irrigation engineers for a long time, that "drainage solves the alkali problem", is no longer thought to be true in all cases. So many failures have been recorded that the question of alkali reclamation is still in the doubtful stage. It is believed that the time will come when alkali land will be reclaimed with assurance of success. At present, efforts can be best directed toward the prevention of the accumulation of alkali.

Engineering Studies Preliminary to Drainage

Ground-water follows the well-known and definite laws of hydraulics in moving down grade. The rate of movement is not so well understood, as the retarding influence of the material through which the water moves is not known, nor is the result of a number of factors, only a few of which can be evaluated. The variable nature of the strata also complicates the problem. The result is that there is a great deal of estimating in any effort to trace the direction, rate, or amount of underground flow.

It is possible, however, to remove much of the uncertainty if there is opportunity to make observations over an extended period of time. If this time includes the period in which the water is moving toward the place where drainage is needed, the solution of the problem can be made with a satisfactory degree of exactness. If no observations are made and water suddenly appears beneath and threatens the future of an area of land, it may be very difficult to determine the source of the water and the effort to remove it must to a certain extent be experimental.

Ground-water studies should be a part of the regular duties of the organization engaged in the operation and maintenance of irrigation works. These studies should be continued with regularity. It is important to know the depth to ground-water and the fluctuations through which the ground-water goes during the season. If, then, the rate of use of water is known and the canal losses are recorded, the drainage engineer will have data on which to base his studies.

Drainage Construction

Construction works to correct drainage problems may take several forms. The important ones are:

1.—Removal of Water at the Source.—This can be done by reducing canal losses or deep percolation losses from irrigation fields. Both these sources are important and frequently they can be reduced or controlled at less cost than by removing the water from the area affected.

2.—Open Canals.—Open drains to effect the reduction of water-plane level by excavating below that level are in common use in many irrigated areas. Cheap excavation by machinery has brought this method into popularity in recent years. The large area occupied by right of way and the annual cost of cleaning are the most important objections to open ditches.

3.—Tile Drains.—Tile drains are effective, but are used mostly in the details of farm drainage rather than in large works of great extent. Engineering methods of tiling are well developed. The first cost is the principal objection to tile. Concrete pipe, which now can be made in large sizes at costs less than clay pipe, has greatly extended the use of large tile drains. The effect of alkali salts on poorly made pipe has been troublesome in some regions.

4.—Cut-Off Drains.—Cut-off or intercepting drains, either open or tile, are often effective in collecting ground-water between source and place where it will cause damage. Where relatively water-tight strata cause the flow to follow overlying porous members within reach of drains, this method frequently proves effective.

5.—Pressure Relief Drains.—Where water is entrapped under relatively water-tight strata and is sub-irrigating under pressure, relief may be obtained by tapping this water by open wells or drains cutting below the confining material. Very simple works are sometimes very effective in improving large areas.

Conversely, to this method, there sometimes are encountered perched water planes which may be drained away by the penetration of the confining stratum by wells or open drains. Local basins or saucer-shaped areas of hardpan or rock in the subsoil, underlaid by porous materials not filled with ground-water, may be relieved by draining through wells.

6.—Wells for Drainage.—Pumping from wells is rapidly becoming recognized as an effective and, frequently, cheap method of removing excess of water. This is a vertical drain with the water removed by pumping. The development of deep-well turbine pumps in which a wide range of head below the pump is possible, has made this method of draining practical.

Where cheap power is available, this method should be considered. It has the advantage of collecting the water in such a way that it can be used for irrigation, the value of the water often paying a substantial part of the drainage cost.

This method apparently has a much wider application than was thought a few years ago. The vertical movement of water through strata seems easier to bring about than lateral movement. The reduction of pressure by pumping from beneath water-tight strata may expose a large area of the material to downward percolation and result in reduction of water levels to a surprising extent.

Drainage by wells has been used to a limited extent in many irrigated areas. It is used to almost the exclusion of other methods of drainage in the Salt River Valley of Arizona and in the South San Joaquin, Modesto and Turlock, and Merced Irrigation Districts in California. In these regions where electric power costs from ½ cent to 1¼ cents per kw-hr., it has been found that drainage water can be removed by pumping from wells at less cost than by any other method.

Thomas H. Means, *Chairman,*
Committee of Irrigation Division on
Drainage of Irrigated Lands.

DISCUSSION

M. R. Lewis,* M. Am. Soc. C. E. (by letter).—This report is of great interest and, as a brief summary of the subject, is certainly of permanent value. It shows careful thought and a broad understanding of the problem. However, there are a few instances in which the writer believes it should be modified or elaborated.

In the section devoted to the "Cause of Necessity for Drainage", the influence of alkali is omitted. The almost universal presence of excessive quantities of soluble salts in irrigated areas necessitates much deeper drainage than is common in humid areas. Unless the water-table is lowered to such a depth that only negligible quantities of water will be brought to the surface and there evaporated, a concentration of alkali salts will occur. Where the drainage is shallow and, as a result, the water-table is only 2 to 4 ft. below the surface, the rise of alkali can be controlled only by complete flooding of the surface by irrigation water.

In most sections this water carries in solution comparatively large quantities of soluble salts. Frequently, the quantity of most of these salts exceeds that carried off by the harvested crops. As a result, in the absence of all downward percolation to the water-table, there will be a gradual concentration of alkali salts in the surface soil. Even where the water-table is at a satisfactory depth below the surface, this fact makes it essential that some water be allowed to percolate downward through the soil to the ground-water. This water also must be removed by under-drainage, either natural or artificial. For this reason, even if it were possible, it would not be advisable to irrigate with exactly the quantity of water which is used by crops.

In the discussion of the water-holding capacity of irrigated soils, two factors have become confused. Tables 3 and 4 purport to give the capacity of various types of soils to retain moisture. Actually, they give some combination of the capacity to retain moisture and the ability to absorb it under field conditions. The water-retaining power of soils is progressively greater as their texture becomes finer. (See Fig. 1.†)

There is considerable evidence that the moisture equivalent is a very fair measure of the field capacity of a soil, both relatively and absolutely. While it is difficult to demonstrate this fact in heavy clay soils on account of their impermeability, the conclusion seems reasonable that fair agreement will be found, at least in surface soils which are not under considerable pressure.

It is true that under field conditions it is difficult if not impossible to make the heavier clay soils absorb as much as they can retain. However, it is best to keep clearly in mind that this is a problem of application of water and not one of its retention. As given in the report, Table 4 would indicate almost as great danger of over-irrigation of clays as of sands. In fact, of course, there is no such danger.

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* Irrig. and Drainage Specialists, Oregon State Coll., and Div. of Agri. Eng., U. S. Dept. of Agriculture, Corvallis, Ore.

† Adapted from "Outline of Ground-Water Hydrology," by O. E. Meinzer, U. S. Geological Survey, *Water Supply Paper No. 494*.

In the reclamation of black alkali lands both the Oregon and the Idaho Agricultural Experiment Stations have secured promising results by planting sweet clover on the undisturbed surface of the ground and irrigating very frequently. As stated in the report, further studies of this problem are needed, and they are being made.

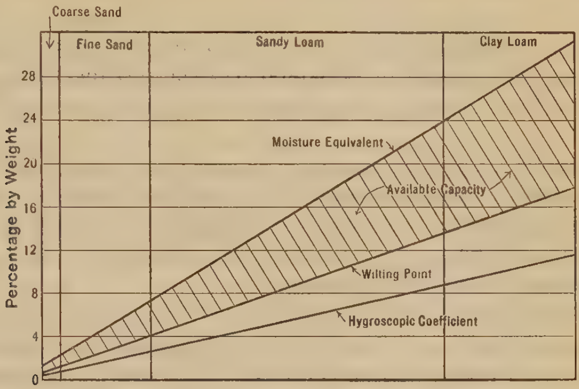

FIG. 1.—MOISTURE-HOLDING CAPACITY OF SOIL.

Control of the use of water by the farmer is a more complicated problem than is indicated in the report. Not only must the capacity of the soil to hold moisture be known, but also its ability to absorb that water. In tests on alkali lands the writer found that at the end of 1 hour the soil at one spot absorbed water at the rate of 0.16 ft. per hour, whereas at a point 75 ft. away, in the same border strip, the rate was 0.019 ft. per hour. Other tests on a supposedly uniform loess soil at Moscow, Idaho, showed a rate of infiltration three times as fast at one point as at another 5 ft. away. The problem of the rate of infiltration and application to the field of the facts discovered is a pressing one.

The use of pumping for drainage appeals to the writer as being the greatest single forward step made by irrigation engineers in many years. He expects to see its application very greatly extended, primarily because of the apparent ability to lower the water-table by this method to much greater depths than is feasible by either open or closed drains of the gravity type.

In this connection it may yet be found that "drainage solves the alkali problem" at least to a very great extent; but it must be real drainage, not just a feeble lowering of the water-table below the surface.

MEMOIRS OF DECEASED MEMBERS

FAYETTE SAMUEL CURTIS, Past-President, Am. Soc. C. E.*

Died February 15, 1929.

Fayette Samuel Curtis sprang from early American stock. Henry Curtis, his paternal ancestor, came from England in 1635, settling first at Watertown and then at Sudbury, Mass.; his mother's family also were early settlers of that State.

Fayette Samuel Curtis, the son of Allen and Catherine (Steel) Curtis, was born on December 16, 1843, at Owego, N. Y., and lived there for the first twenty years of his life, attending the district school until he was twelve, then private school for two more years, and, finally, Owego Academy, where he received special instruction in Civil Engineering.

Finishing with Owego Academy in 1863, Mr. Curtis was employed on the location and construction of a line for the Albany and Susquehanna Railroad Company between Binghamton and Albany, N. Y., first as Rodman and, then, in 1864, as Transitman. Two years later, he went with the Southern Central Railroad Company on its line from Owego to Auburn, N. Y., serving as Levelman during the location, and then, with the commencement of construction, as Assistant to the Resident Engineer. On the completion of this work he accepted a position as Transitman with the Lake Ontario Shore Railroad Company on location between Oswego and Charlotte, in New York State.

In the spring of 1869, as Assistant to the late George S. Greene, Jr., M. Am. Soc. C. E., young Curtis was employed on surveys in the Towns of Morrisania, Tremont, and Fordham, now parts of Greater New York, which surveys have since served as the base of much of the later mapping in The Bronx. After two years of this work, in April, 1871, he returned to the railroads, as Assistant Engineer, under the late I. C. Buckhout, M. Am. Soc. C. E., on the New York and Harlem Railroad, and six months later, on October 1, he became Principal Assistant, first on the location of the tracks and yards at the old Grand Central Station, in New York, N. Y., and, later, on the plans and the construction of the Fourth Avenue Improvement between the Grand Central Station and the Harlem River.

On the retirement of Mr. Buckhout in 1874, Mr. Curtis became Chief Engineer of the Harlem Railroad Company and Superintending Engineer of the Fourth Avenue Improvement, continuing in the service of the former Company until the completion of its line. The Harlem Railroad, about this time, was leased to the New York Central and Hudson River Railroad Company, and while Mr. Curtis was still acting as Engineer for the Harlem Lines, the grain elevators, yards, docks, piers, sheds, and warehouses on the Hudson River, between Fifty-ninth and Seventy-second Streets, were designed and built for the New York Central under his direction.

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* Memoir prepared by Clarence Blakeslee, M. Am. Soc. C. E., assisted by E. J. Beugler, C. C. Elwell, Edward Gagel, H. C. Keith, H. L. Ripley, Henry B. Seaman, Paul Sterling, and E. W. Wiggin, Members, Am. Soc. C. E.

On December 1, 1882, Mr. Curtis was called to the New York, New Haven, and Hartford Railroad Company, as Chief Engineer, with headquarters first at New York, and later at New Haven, Conn. This position he held until May 1, 1900, when he became Fourth Vice-President in charge of "engineering and such other matters" as might be assigned to him by the President or the Board of Directors, his headquarters being at Boston, Mass.

As of June 1, 1903, his duties were more specifically set out as follows: "General supervision of the Engineering Department and construction, including the Electrical Department, except operation, also maintenance of way, and construction and maintenance of buildings and signals, except the operation of signals". This assignment remained unchanged when, on November 14, 1903, Mr. Curtis was made Second Vice-President, with headquarters at Boston, and also when, a year later, he was transferred to New Haven. As of October 1, 1905, however, he returned to Boston, to "represent the President with full authority in all matters requiring his attention", and remained in this position for two years. On April 1, 1907, he resigned the Vice-Presidency of the New York, New Haven, and Hartford Railroad Company to accept the Presidency of the Old Colony Railroad Company, which latter position, together with those of President of the Union Freight Railroad Company, of Boston, and Chairman of the Boston Terminal Company, he held at the time of his death.

Mr. Curtis came to the New York, New Haven, and Hartford Railroad Company very largely through the influence of Mr. Charles P. Clark, then Second Vice-President, who already had well in mind plans for a unified transportation system for Southern New England, and who correctly appraised the worth of Mr. Curtis as an assistant in the development. The latter had but just come to the property, however, when Mr. Clark accepted a position with the New England Railroad Company and the next four years in the engineering affairs of the New York, New Haven, and Hartford Railroad Company were marked merely by consistent rather than by extensive improvements.

In the spring of 1887, however, Mr. Clark returned to the New York, New Haven, and Hartford Railroad Company, as President, and immediately began to build up and to expand the property. In this work Mr. Curtis played a most important part as one of the group of trusted lieutenants who served not only in the fields specifically assigned to them, but as general advisers and assistants as well. How highly his judgment was esteemed is shown by an endorsement on a letter in the office files of the Company. President Clark had referred an inquiry to Mr. William E. Barnett, then Executive Secretary and Attorney, who returned it with the note: "I do not know; ask F. S.; he has second sight if any one has."

In December, 1882, when Mr. Curtis came to the New York, New Haven, and Hartford Railroad Company, the line had double track from New York to Springfield, Mass., approximately 123 miles, and single track elsewhere, the total single-track equivalent being but a little more than 400 miles. This was laid with 60-lb., or lighter, rail, most of the latter being iron. The great majority of its highway crossings were at grade, and in the year ending September 30, 1882, it had operated in round figures 185 000 000 passenger-miles and had moved 117 000 000 ton-miles of freight.

By 1907, the line from New York to New Haven had been completely four-tracked, as had that part of the Providence Division from Readville into Boston, so that at the close of 1907, when Mr. Curtis' assignment to Boston as "representative of the President, with full authority in all matters requiring his attention", made it desirable to relieve him of close control of engineering, there were being operated about 4 254 single-track equivalent miles with $89\frac{2}{3}$ miles of line four-tracked and $651\frac{3}{4}$ more, double-tracked, largely laid with 100-lb. rail on stone ballast. The section of this rail was designed by Mr. Curtis in 1893, its height of 6 in., with a 6-in. base, giving extra stiffness and stability. There was not a single grade crossing on the four-track sections, from New York to New Haven, and that in the vicinity of Boston, and a large number elsewhere on the System had been eliminated, notably in connection with very extensive improvements at Westerly and Providence, R. I.; Hartford, Conn.; and Attleboro, Brockton, Fall River, and Worcester, Mass.; while the traffic had so increased that in 1907 there were operated roundly, 1 256 000 000 passenger-miles, and 1 889 000 000 ton-miles of freight were moved.

This, however, is but the merest outline of the story of railway improvements for which Mr. Curtis largely was responsible. With much of the work in thickly settled communities, and all of it adjacent to, if not directly under, heavy traffic, engineering ability of the highest quality was demanded, both of the Chief and of his assistants. How well the test was met by Mr. Curtis' professional skill and the keen discernment with which he picked his lieutenants, is clearly shown in the fact that in the conduct of this great work not a single passenger was killed or seriously injured; and this record was practically duplicated as regarded the general public, except as to accidents resulting from the personal carelessness and disregard of barriers and warnings by the injured persons.

Nor is it without significance that, except as in some instances prospective increased loadings far above those for which they were designed have necessitated the replacement of some of the earlier structures, there have been neither failures nor indications of weakness in any of the structural work. This required that every contractor be held rigorously to his agreed performance, but it is quite characteristic of the fairness with which Mr. Curtis maintained his standards that in not a single instance was there an appeal to higher authority, either within the Company or without, relative to his decisions.

Carrying to an extreme his dislike for any publicity, there is on record very little regarding the engineering features of this work thus broadly outlined. To the engineer familiar with New England conditions, however, the bridges, particularly over the tidal estuaries and the larger rivers, the elevated work through Bridgeport, Conn., Attleboro, Brockton, and the approach to Boston, the "cuts" at Mount Vernon, N. Y., New Haven, and in Boston; the stations at Bridgeport, Hartford, and Providence, as well as many less important—all tell unmistakably of the hand of the master engineer.

Mr. Curtis also played an important part in the design of the South Station, Boston. He, with L. B. Bidwell, Chief Engineer of the New York and New England Lines, and Walter Shepard, Chief Engineer of the Boston and Albany Railroad Company, representing the three roads for which the station was built, constituted the Consulting Engineering Board for that project. In 1895, Mr. Curtis accompanied President Clark and a party of officers of the New York, New Haven, and Hartford Railroad Company to the Fifth International Railway Congress in London, England. At the close of the Congress he, Mr. Bidwell, and Mr. Shepard, visited the more important railroad stations of England and the Continent, studying their operation, their advantages, and their shortcomings, later employing the knowledge thus secured to the great advantage of Boston.

"F. S." as he was very generally called, was a man of tremendous energy and splendid constitution. Up to less than a year before his death the average man of 65 would have found it very difficult to keep up with him. This was the more remarkable because he had never spared himself. His "day" ended when the work permitted, without reference to the time, and during the lifetime of Mr. Clark a long active day in the field or office frequently was followed by conference until early morning with his equally tireless President.

Possessed of a kindly disposition and a keen sense of humor, Mr. Curtis had the happy faculty of making and keeping friends. His consideration of, and advice to, the young engineers who came under his direction were such as to inspire them to high ideals of engineering accomplishment, and there are many one-time associates who owe their later successes to his early training. With all his duties and responsibilities he always found a minute in which to greet even casual acquaintances, and to exchange the latest stories, his hearty and contagious laugh telling his keen enjoyment of a good tale. He was very fond of horses, and was a regular patron of the old Morris Park Race Track, taking unusual pleasure in attending the races with a crowd of his particular cronies.

He was a member of the Connecticut Society of Civil Engineers, which he had joined on January, 9, 1900, and of which he was made an Honorary Member on February 19, 1924.

Socially, he was a member of the Beacon Society, the Boston Athletic Association, and the Engineers Club of New York; his membership in the Quinnipiack Club of New Haven, which he had long held, he resigned about a year before his death.

Fraternally, Mr. Curtis was a prominent Mason, holding membership in Friendship Lodge, Harlem, in Jerusalem Chapter, of Tioga County, and in Constantine Commandery, Knights Templars, all in New York State.

On October 19, 1872, he was married to Mary J. Bishop, of New Haven, who survives him, as does a sister, Kate S. Curtis, of Owego, and a half-brother, Robert Curtis, of New York. Of his three children, Wallace, George Henry, and Mary J., only the latter, who is the wife of Capt. John D. Woodworth, Medical Corps, U. S. A., at Vancouver, Wash., is now living. There are, however, five grandchildren: John Fayette, son of Wallace; George Henry and Dorothy M., children of George; and Fayette Curtis and Mary Justin, children of Mrs. Woodworth. Mr. Curtis' death, after an illness of three months, occurred on February 15, 1929, at his home in Jamaica Plain, Boston.

An engineer and executive of outstanding knowledge and ability, an indefatigable worker, a staunch and loyal friend, and a gentleman, his death is a great loss to his friends, to his profession, and to all who had the privilege of being associated with him or knowing him.

Mr. Curtis was elected a Member of the American Society of Civil Engineers on April 3, 1889. He served as Director from 1895 to 1897, inclusive; as Vice-President in 1904 and 1905; and as President in 1919.

CHARLES MACDONALD, Past-President, Am. Soc. C. E.*

DIED JULY 8, 1928.

Charles Macdonald was born at Gananoque, Ont., Canada, on January 26, 1837. He was the son of W. Stone Macdonald and the grandson of Charles Macdonald, who married the daughter of Colonel Joe Stone, the founder of Gananoque. After attending the public schools of his native town, and, later, a preparatory school of what was then Queen's College, Kingston, Ont., he obtained a position on the surveys of the Grand Trunk Railway between Kingston and Brockville, Ont., on which he served during 1852 and 1853. In 1854, Mr. Macdonald entered Rensselaer Polytechnic Institute, at Troy, N. Y., which was then the only institution of its kind in the United States, and was graduated therefrom with the degree of Civil Engineer in 1857. At the time of his death he was a Trustee of the Institute and its oldest graduate. In later years, he also received the degree of Doctor of Laws from Queen's University, Kingston, Ont.

Following his graduation Mr. Macdonald was engaged on the construction of an extension of the Grand Trunk Railway from Huron, to Detroit, Mich. Little engineering work was done in Canada during the next few years, although Mr. Macdonald lived at Gananoque until 1863. In the early spring of that year he moved to the United States, taking a position in charge of the Engineer Corps of the Philadelphia and Reading Railroad Company, with headquarters at Pottstown, Pa. Shortly afterward, he joined the Pennsylvania Volunteers, advancing to the rank of Corporal. He was made a prisoner in the Battle of Gettysburg.

Following the close of the Civil War Mr. Macdonald devoted much time to the study of iron bridge construction which was then in its infancy. A pamphlet on this subject which he wrote, and which found favorable attention, caused him to be entrusted with the construction of the bridges in the extension of the Delaware, Lackawanna and Western Railroad, to Hoboken, N. J.

In 1869 he opened an office in New York, N. Y., devoting himself to the design and construction of iron bridges. Mr. Macdonald was also much interested in the development of swing bridges. In 1871-1872 he designed and built the Point Street Bridge† in Providence, R. I., which was in continuous service under constant traffic until it was replaced by a heavy, wide, modern structure during 1927. He was appointed by the City of New York as a Trustee of the East River Bridge, of which Colonel Washington A. Roebling was Chief Engineer, which was opened in 1883.

In 1884, Mr. Macdonald organized the Union Bridge Company (of which he later became President), with headquarters in New York and shops at Athens, Pa. Of his partners, George S. Field, M. Am. Soc. C. E., and the late Edmund Hayes, M. Am. Soc. C. E., had their offices in Buffalo, N. Y.; and the late Charles Kellogg and Charles S. Maurice, Members, Am. Soc. C. E., in

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* Memoir prepared by a Committee consisting of W. G. Triest, M. Am. Soc. C. E., *Chairman,* the late Clemens Herschel, Past-President and Hon. M., Am. Soc. C. E., and O. E. Hovey, M. Am. Soc. C. E.

† Description appears in *Engineering,* Vol. XV (1873), p. 202.

Athens; that of the late Thomas C. Clarke, M. Am. Soc. C. E., was in New York. The Union Bridge Company built many large bridges in the United States, including the Poughkeepsie Bridge, erected in 1887 and 1888. The Company also designed and built the Hawkesbury Bridge, in New South Wales, Australia, during 1886 and 1887. Both these structures had long spans and unusually deep foundations.

He was one of the earliest promoters of a suspension bridge over the Hudson River at 57th Street, New York. A large model of this bridge was on view in his office. He was active in the preliminary work and the organization of the American Bridge Company, to which he sold the Union Bridge Company in 1900; he served as one of the Vice-Presidents of the former company until the following year. He then retired from active business, but was occasionally engaged in consulting work, notably in 1910, as one of the Commissioners of the Quebec Bridge during its reconstruction.

Mr. Macdonald was very much interested in the growth and affairs of the Society, and contributed frequently to its *Transactions*.* He also discussed many papers presented by others, particularly those relating to bridge construction and materials. He was an early advocate of the use of steel in place of wrought iron in bridges and during 1879 proposed the use of high carbon steel for eye-bars and suggested the practicability of making steel eye-bars having a yield point of 63 000 and an ultimate tensile strength of 106 000 lb. per sq. in., which has been accomplished during the past few years.

Socially, he was much interested in the Engineers' Club, of which he was made an Honorary Member, the Century, the Lawyers', the University, and Union Clubs, of New York, and the Engineers' Club, of Montreal, Que., Canada. Mr. Macdonald was accustomed to divide his summers between his home at Rumson, N. J., where he was one of the first summer residents, and his place "Blinkbonnie", at Gananoque, on the shores of the St. Lawrence River.

In 1861, he was married to Sarah Louise Willard, the daughter of Colonel Willard, of Troy, N. Y. There were three children—a son, William Stone Macdonald, and two daughters, Mrs. John Serson and Mrs. W. G. Triest. After his wife's death in 1912, Mr. Macdonald made his home with his children, but he decided to return to his native town in Canada in 1926, where he spent his last years. He died there on July 8, 1928.

Mr. Macdonald was elected a Member of the American Society of Civil Engineers on September 15, 1869. He served as Director in 1871, and from 1874 to 1876; as Vice-President in 1893 and 1894; and as President in 1908.

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* "Proportions of the Heads of Eye-Bars," *Transactions*, Am. Soc. C. E., Vol. II (1872-73), p. 333; "The Failure of the Ashtabula Bridge," *Transactions*, Am. Soc. C. E., Vol. VI (1877), p. 74; "The Ocean Pier at Coney Island," *Transactions*, Am. Soc. C. E., Vol. VIII (1879), p. 227; "The Six-Hundred Ton Testing Machine at the Works of the Union Bridge Company at Athens, Pa.", *Transactions*, Am. Soc. C. E., Vol. XVI (1887), p. 1; and "Presidential Address at the 40th Annual Convention at Denver, Colorado, June 23d, 1908," *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 544.

FERDINAND FOCH, Hon. M. Am. Soc. C. E.*

DIED MARCH 20, 1929.

Ferdinand Foch was born on October 2, 1851, at Tarbes in the Department of Hautes Pyrénées, France. Although his ancestry was not illustrious, he was sprung from the class of country gentry, a class that always forms the backbone and main strength of every people, and particularly of the French. His father was a practising lawyer, a calling which, however, did not appeal to the son, whose inclination was toward mathematical pursuits, although his strong religious sentiments at one time led him to consider taking Holy Orders. His brother did so, becoming a priest of the Jesuit Order, but Ferdinand was entered in 1869 at St. Clément's College, Metz, to study for the Ecole Polytechnique.

The Franco-German War broke out the year following and young Foch promptly volunteered, but by the irony of fate the future Maréchal of France and conqueror of the Teutonic armies saw no active service, being drafted into a unit that was held in Central France away from the fighting line.

The war ended; M. Foch, discharged from the army, took his examinations for the Ecole Polytechnique, from which he was graduated, commissioned, and assigned to the Artillery in 1873. After a short service with troops he was attached to the General Staff, and, in 1895, was appointed a professor at the Ecole de Guerre, when his military rank was that of a Lieutenant-Colonel. In 1901, he was named Director of the Ecole, serving until 1905.

There is a pretty story that M. Clemenceau, then President of the Council, appointed Colonel Foch to the post without any previous knowledge on his part that he was being at all considered. Unfortunately, this story lacks confirmation. Colonel Foch was a candidate, believing that his career lay in teaching, and he was given the post after long and careful consideration. This was one of M. Clemenceau's greater acts. Colonel Foch not only had no political influence, but had incurred the hostility of the radical party then in power on account of his strong religious views and devout practice. M. Clemenceau made the selection of Colonel Foch solely on the score of merit.

While a professor he delivered a series of lectures on the Art of War, which have been published in two volumes under the titles of "De la Conduite de la Guerre" and "De les Principes de la Guerre", and have been translated into many languages. In the latter volume he analyzed the campaigns of Napoleon, who was his military idol and inspiration; and in the former, the major battles and strategy of the Franco-German War of 1870. These lectures are particularly interesting as he shows that, with only slight improvement in French strategy, the German plan of campaign would have broken down. For these conclusions he gave full authority by quoting official German orders.

In 1911 he was given command of the 13th Division of Infantry, the first body of troops that he had commanded since as a Captain he had been at the head of a battery of artillery.

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* Memoir prepared by Wm. Barclay Parsons, Hon. M. Am. Soc. C. E.

When General Foch was 63 years old, or within a year of the time when an American army officer would be automatically retired "for age", the World War broke out. At that time he was commanding the Twentieth Army Corps with headquarters at Nancy, France. In July, 1914, he had been given leave and was visiting his country place, the Château de Transfeuntenisu, near Ploujean, on the Brittany Coast, to make ready against the enforced longer leave that he must soon take, when he received a peremptory order to return at once to his command. A few days later Europe was in ferment.

With the organization of the greatly enlarged French Army, following mobilization, there was a demand for officers to assume high and important rank. General Joffre had been made Commander-in-Chief of the French forces, and selected General Foch to command the Ninth Army. Thus, there was presented the extraordinary spectacle of an Army Commander, 63 years of age, whose military service had been almost exclusively confined to work on the General Staff, or in the lecture halls of the Ecole de Guerre. He had taken no part even in the small wars that France had carried on in the Colonies, nor had he ever handled large bodies of troops in the field. His experience had been wholly theoretical and didactic.

When General Joffre, while fighting masterly rearguard actions during August, 1914, in the retreat from Northern France, finally succeeded in placing his forces in such orderly arrangement along the Marne as would enable him to make a stand and by a counterstroke to attempt to seize the offensive, the Ninth Army occupied that portion of the line from Sezanne to Mailly le Camp with headquarters at Fère Champenoise. On the left was the Fifth Army commanded by General Franchet d'Esperey, and on the right the Fourth Army under General Langle de Cary. Opposed were parts of the Second and Third German Armies with Generals von Bülow and von Hausen.

On September 6, 1914, when the order was given to the whole line to stop retreating and to advance, no one saw more clearly than he, who had taught the theory of war, what the whole situation presented. If an advance could not be made, a further retreat with defeat was inevitable, while, on the other hand, an advance that did not completely dislocate the enemy's plan of attack would not suffice to avert impending disaster. For two days the German forces were thrown against the French and British line to break its resistance, and nowhere was the attack heavier than on the front of the Ninth Army. So violent was the pressure that General Foch had been forced to yield ground.

In his lectures he enunciated the principle that there was such a thing as the "Will to Victory", deducing from it that a battle is lost only when a commander believes himself beaten. On the afternoon of September 8, with his own resolve to win firmly fixed in spite of local reverses, he saw that the enemy forces had been drawn into an unfavorable position, which could be availed of by a rapid and daring shift of his own troops. It was then that he sent his famous dispatch, "My right heavily pressed, my center yields, situation excellent, I attack." It was the temporary faulty position of the forces in front of him that presented the excellent situation. This message was intended to cheer the drooping spirits of men who had been in continuous retreat for four weeks and who had but recently come under his command.

General Foch attacked with such vigor that the enemy, whose extreme right wing was in difficulty, saw that an immediate retirement along the whole front was necessary to avert a disaster. Paris and France were saved.

When time has removed all personal bias and an impartial study can be undertaken to apportion credit and evaluate effect between the genius of Joffre and Gallieni, the surprise pressure that developed on the enemy's right, the fact that the enemy had outrun his reserves and supplies, and other causes contributing to the victory on the Marne, there would seem to be no question that General Foch's brilliancy in discerning an "excellent situation" in the midst of what appeared to be a crushing defeat, and his alertness in seizing it, will be ranked as one of the most potent factors.

With any further advance temporarily stopped the Germans endeavored to correct an initial error which they had committed when they left the channel ports free. To assist in their defense by co-ordinating the efforts of the British, French, and Belgian forces, General Foch was sent to Northern France with the title of Commander of the group of Armies of the North. His authority was limited, however, to the French forces, and his relations with respect to the British and Belgian Allies were those of a liaison officer. In this double capacity he remained until the end of 1916, taking prominent part in the Artois and Somme offensives, during which there was some of the bloodiest fighting in the whole war.

Then, through one of those unfortunate developments in internal politics, which have no relation to the conduct of war, but which in a democratic government seem to be inseparable from it, General Foch was sent into temporary retirement, being ordered to Senlis "to rest", although later given nominal inspection duty and ordered to prepare a plan for resistance against a possible German invasion of Switzerland. In 1917, he was made Chief of the General Staff with headquarters in Paris, a position that carried no field command.

On March 21, 1918, the Germans began their last and great offensive. A wedge was driven between the British right and the French left just south of Peronne, involving the retirement of the British Fifth Army and the withdrawal of the French left. Not only was the whole area of the Somme, that had taken the Allies five months in 1916 to capture at appalling cost, regained in as many days, but additional territory was seized that had been held by the British or French since 1914. The enemy was in front of Arras and Amiens, and the main lines of communication from Paris northward were either cut or rendered useless as being under fire.

The situation was as desperate as it was on the Marne in 1914, and it was obvious that some drastic step must be taken to prevent a break-through to the Channel that would divide the Allies into two parts and involve disaster. As Field Marshal Haig put it, "We are fighting with our backs to the wall." There were three armies in the northern field co-operating but not correlated, French, British, and Belgian. There was a fourth, the American, just entering the field. These armies were under their own individual commanders, who conferred and acted jointly as best they could, while preserving their own separate authority. They "loaned" each other troops and supplies, if sufficient emergency could be shown, but in accordance with the dictates of human nature, each commander reserved to himself the right to decide on the seriousness of the other's emergency and was reluctant to release reserves or material for fear that he might immediately find himself, in turn, in sore straits.

For three years it had been patent that this division of control was a serious handicap as opposed to the single will presented by the enemy, but as yet no nation was willing to accept an uncontrolled command vested in another national. Victory and defeat were now in the balance, and if the former were to be secured all personal antagonisms must be submerged into the greater consideration of the welfare of the common cause.

On March 26, 1918, a conference between the leading French and British military leaders and ministers representing the respective War Departments was held in the Hôtel de Ville, of Doullens, a small village about 20 miles north of Amiens and directly behind the front line, to determine a policy of joint action. It was at once agreed that complete unity in command could be postponed no longer, and the sole question was to find the man. Various names were proposed, even that of M. Clemenceau being brought forward, but after long discussion only General Foch had not been rejected. Sir Douglas Haig stated that he would accept Foch as Generalissimo and give him full support, an offer immediately repeated by General Pétain, then the French Commander-in-Chief. M. Clemenceau had General Foch waiting, because from the outset he was M. Clemenceau's candidate. He was called in and accepted the burden, so that finally after three and one-half years of blundering, a single directing force faced the common enemy. A few days later, General Pershing pledged the new Commander-in-Chief his support and that of the American Army, provided only that the latter should be used as a unit.

There is no parallel in history to the task that General Foch undertook that night in spite of his 67 years. The French, British, and Belgian Armies totaled nearly 10 000 000 men. They were wearied with almost four years of continuous fighting, during which time each had learned to think of itself as a separate unit working to some extent conjointly with others. Recently, they had received a blow that had sent them reeling and one that had nearly caused collapse. Guns in great number and material in huge quantities had been lost. Other blows were certain to follow. Could this great mass be welded into a single unit capable not only of withstanding the further attacks that were to come, but of assuming the offensive and striking back? That was General Foch's task, one that called for the stoutest "Will to Victory".

To-day is still too close to those great events when defeat was turned into victory, properly and fully to evaluate the service rendered by General Foch, and the manner in which he accomplished the task that had been assigned to him. To what extent he accepted and was guided by the advice of his Lieutenants, Haig, Pétain, and Pershing, is not as yet clear. Probably he left many details to each Commander, asking only that separate movements should conform to the general plan. His great accomplishment, as to which there will probably be no question, was his success in amalgamating four armies into one, enabling him to treat reserves as one mass, which, whether in men or material, could be diverted or concentrated in such manner or place as he deemed most important. Yet that accomplishment meant victory, and alone justifies the enduring gratitude of the allied nations, of which America was one, and entitles General Foch to be accorded lasting fame. He has described himself as "No more than the conductor of the orchestra"; but what an orchestra!

The next attacks already planned by the enemy could not be averted, but by skillful handling of his forces, General Foch resisted them and gradually brought the offensive to a halt, with Arras, Amiens, and other strategic points safely held. Relieved of the pressure of defending, he set about building up a united army that would be capable of taking a successful offensive. By midsummer his preparations were complete, and he planned to strike the enemy on July 18, in the direction of Soissons.

German headquarters had learned of the proposed attack through their spy system and hoped to escape it by attacking first. This they did early in the morning of July 15, and succeeded in crossing the Marne between Château-Thierry and Dolmans, to a depth of five or six miles. At first, it looked as if the enemy had succeeded in his move, but General Foch, after a careful examination of the situation, decided that the result placed the Germans in a more difficult situation than it did the Allies and decided to continue with his own plan unaltered, leaving but a sufficient force to restrain the enemy from making a further advance. He saw that they had exposed a weak flank, and that his previously prepared plan would compel them to withdraw across the Marne, or have their advanced troops cut off, and this without any sacrifice on his part. This decision well illustrates his method of carefully weighing all the factors at the time of a crisis and not allowing his judgment to be warped either by an excess of caution or a disinclination to change a predetermined plan, and, having reached a decision, to act with firmness. In this particular case it took courage to leave the enemy in his advanced position. It was this ability to think clearly and then to execute without flinching or compromising, that gained for General Foch the unwavering confidence of all ranks.

General Foch's attack was successful and was quickly followed by others in succession until the offensive was general from the Channel to Metz. Steadily under the direction of the master mind the wave rolled on until at the end of October the enemy, realizing defeat, opened negotiations for an armistice, which was signed on November 11, 1918.

In fixing the terms of this armistice, General Foch resisted what must have been a severe temptation for a personal satisfaction. Great pressure was brought to bear on him to continue the war until every German soldier had been driven from France, to sign the treaty on German soil, and to lead a triumphal procession through Berlin. It would have been easy to find justification for such a course, especially as it was supported by popular clamor. The enemy was already in retreat, indicating that his power of resistance was broken, and his own recollection of the King of Prussia being crowned German Emperor at Versailles in 1870 and of the march of the victors down the Champs Elysées, must have been in his mind. He rejected all thought of personal aggrandizement or unnecessary risk, however, arguing that there was no justification in prolonging the war one hour or shedding one drop of blood after the main objective had been gained, which, in this case, was the expulsion of the invader from France.

The war ended, General Foch made no effort to secure political advancement. He was a soldier fighting when his country needed his services and was not of the type of the "Man on Horseback" that France had always feared would come into being following a victorious war against Germany when the "stolen" Provinces would be returned. As soon as his military duties permitted, he withdrew to his estate of Transfeuntenisu.

All nations vied in heaping upon him every honor available. France made him a Maréchal and the Academy elected him as one of 40 "Immortals." Great Britain created him a Field Marshal and conferred on him the Grand Cross of the Order of the Bath and the Order of Merit. So it was with every nation associated with the Allied cause.

In 1921, he made his first and only visit to the United States. On December 6, he was received in the Engineering Societies' Building and was presented for Honorary Membership in all the Founder Societies, the only man who has ever received such distinction as a joint award.*

During 1928, his health began to fail and, although he made the same determined fight against the one enemy who cannot be beaten, he was able only to postpone the inevitable. He died at his residence in Paris, on March 20, 1929. He was buried in the Invalides on March 26, 1929, close to his great hero, Napoleon.

Ferdinand Foch was married to Julia Bienvenue, by whom he had one son and two daughters. His son, an officer in the French Army, was killed at the outbreak of the war (August, 1914) in the Ardennes. It is related that when word of his son's death was brought to him he requested the members of his staff to withdraw and leave him alone for half an hour. When they returned they found him at his table studying his maps and prepared to carry on.

He was of medium height with a well-made frame. His face, although it showed energy and determination, had in repose a gentle, kind expression. His whole appearance suggested the man of action, but perhaps that of the man of affairs rather than the type of Roman conqueror that popular fancy has in mind for the military leader.

Marshal Foch was elected an Honorary Member of the American Society of Civil Engineers on December 6, 1921.

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* For an account of the presentation see *Proceedings*, Am. Soc. C. E., January, 1922, Society Affairs, pp. 4-7.

SAMUEL REA, Hon. M. Am. Soc. C. E.*

Died March 24, 1929.

Samuel Rea was born in Hollidaysburg, Pa., on September 21, 1855, the son of James D. and Ruth (Moore) Rea, and the grandson of General John Rea, of Chambersburg, Pa., who was an Officer in the Revolutionary War.

Before reaching the age of sixteen, Mr. Rea entered the service of the Pennsylvania Railroad Company as a Rodman in one of its Engineering Corps. With the exception of a few short intervals, he served in that organization during the remainder of his life as Engineer, Principal Assistant Engineer, Assistant Second Vice-President, Assistant to the President, First Assistant to the President, Fourth Vice-President, Third Vice-President, Second Vice-President, First Vice-President, Vice-President and, finally, President from January 1, 1913, to September 30, 1925. On the latter date he retired as President under the Pension Regulations. Mr. Rea was the only President to attain the age of 70 years in active service, none of his predecessors in that office having lived beyond 63 years.

His employment with the Pennsylvania Railroad Company and his active and vigorous interest in it through the period of its ever-increasing development and importance as the leading transportation system of the country, were the heart and soul of his existence—in fine, he lived the Pennsylvania Railroad.

His quick grasp of difficult questions, his exceptional memory, his natural ability to analyze and solve railroad problems, and the opportunity of growing up in close touch with the development of the large area served by the Pennsylvania System, all welded together in him an amazing knowledge of the property and the territory it serves. It may truthfully be said that he knew more about the physical and financial characteristics of the Pennsylvania Railroad, its history and problems, than any of his contemporaries.

The range of Mr. Rea's activities in the service of the railroad was extensive and diverse. Early in his career he centered his attention on the numerous branch lines of the Pennsylvania System which, in his case, meant not only their construction and operation, but also their corporate formation, financing, etc.

More than thirty years ago he foresaw the desirability and need of railroad consolidations as a factor that would contribute toward more efficient and economical operation of the railroads. The Pennsylvania System originally consisted of more than 600 constituent transportation companies, which at the close of Mr. Rea's administration in 1925, and largely through his intensive efforts, had been reduced through consolidations and acquisitions to 127 companies. His activities in this direction gave him the nickname among his associates of "Consolidation Sam."

He was connected with practically all these subsidiaries in the capacity of President or Vice-President, or as a Director. The acquisition of an interest in a large number of lines, such as the Long Island Railroad, the Norfolk and Western Railway, and others, was carried out under his direction.

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* Memoir prepared by W. W. Atterbury, M. Am. Soc. C. E.

His close association with former Presidents Roberts and Cassatt, and former Financial Vice-President John P. Green, served to broaden his outlook and increase his knowledge in many directions in the railroad field.

In the course of his work, Mr. Rea made the acquaintance of Gustav Lindenthal, M. Am. Soc. C. E., the noted bridge engineer, with whom his activities brought him in close association for many years. Mr. Rea took a deep interest in the organization of the North River Bridge Company, of which he was one of the incorporators. The construction of this bridge would have provided an all-rail entrance into the heart of New York City. Failing to secure needed co-operation of other rail lines to assure the success of such a large project, the Pennsylvania Railroad Company proceeded independently to make its own plans for such an entrance by means of tunnels under the Hudson and East Rivers, and Mr. Rea was given direct charge of this work by former President Cassatt. This was most appropriate because, upon his appointment as Assistant to the President in 1892, he had been sent to London, England, to make an examination of the underground railroads then constructed and proposed in that city, and subsequently made a report thereon. Before this, in 1887, he had written a book entitled, "The Railways Terminating in London", a comprehensive study of the physical and financial condition of the English railroads. The success of this great tunnel extension, including the erection of the Pennsylvania Station in New York, N. Y., will always be classed as one of his greatest monuments.

He was also the Executive Engineering Head in the construction of the New York Connecting Railroad, including the Hell Gate Bridge, the longest arch span in the world, which connects the Pennsylvania and New Haven Systems and furnishes an all-rail connection for the Pennsylvania Railroad Company from the South and West with New England and Canadian points.

Mr. Rea took a keen interest in civic and welfare work, making liberal contributions of money and personal effort to worthy causes. He favored regional planning for the Metropolitan Area in and around Philadelphia, Pa., and was Chairman of the Regional Planning Federation of Philadelphia Tri-State District. He not only was Chairman of the Committee to raise funds to carry on this work, but also devoted a great deal of time and study to comprehensive planning for the future of this important section of the country. He was a member of the Board of Trustees of Bryn Mawr Hospital and headed the work of securing a large building fund for that institution. He was for many years President of the Board of Trustees of the Bryn Mawr Presbyterian Church and took a prominent part in building its new edifice, giving a great deal of time to the raising of the necessary money and actual construction work. He contributed the cost of the new organ with the understanding that it would contain no mark or plate showing the name of the donor. Fortunately, he lived to see the Church building completed.

He was also a Trustee of the Pennsylvania Museum and, during his life, contributed a valuable collection of old English silver to the Art Museum in

Fairmount Park, Philadelphia. This is one of the most important private benefactions the Museum has received.

Mr. Rea's interest in railroad transportation was not confined to the Pennsylvania System. His frequent trips throughout the country gave him the opportunity to observe methods of construction and operation on other roads, and the knowledge thus gained was of great value to the Government immediately after the United States entered the World War and before the railroads were taken over at the end of 1917. For eight months prior to that time Mr. Rea and the executives of five other roads constituted what was known as the "Railroads' War Board," under the direction of which the competitive activities of all the important railroads of the United States were submerged, and their properties operated as a continental railway system so as to produce the maximum of National transportation efficiency.

At the time of his death Mr. Rea was a Director of the Pennsylvania Railroad Company and of several of its subsidiaries; also a Director of the Southern Pacific Company, the Philadelphia National Bank, the Bank of North America and Trust Company, the Equitable Life Assurance Society of the United States, the Philadelphia Saving Fund Society, the Bell Telephone Company of Pennsylvania, the Norfolk and Western Railway Company, and a member of the Philadelphia Gas Commission.

Although Mr. Rea did not go to college, having secured his knowledge of engineering through actual experience supplemented by keen observation and study at night, and by extensive travel at home and abroad, in later years he received several honorary degrees, including that of Doctor of Science, from the University of Pennsylvania in 1910; Doctor of Laws from Lafayette College in 1916; and Doctor of Science from Princeton University in 1916.

Mr. Rea was elected a Member of the Institution of Civil Engineers of Great Britain in 1891 and an Honorary Member in 1928, and, at his death, was the only American Member so honored.

On May 12, 1926, the Franklin Institute of Philadelphia awarded him the Franklin Medal, "in recognition of his outstanding work in the conception and construction of railroads, their terminals, tunnels, and bridges, and of his eminently successful application of the principles of science, economics and human relations to railway engineering and administration, in which he displayed vision, imagination and courage of high order".

He was married on September 11, 1879, to Mary M. Black, of Pittsburgh, Pa., and was looking forward to celebrating his Fiftieth Wedding Anniversary in 1929. They had two children, George Black Rea, who died in 1908 at the age of 27 years, and Ruth Rea, who is married to George B. Junkin.

Mr. Rea was a man of simple tastes, one who avoided ostentation and was beloved by all. He was devoted to his family and his life's work; he was a great reader and intensely interested in the growth of the country and its industries—a loyal friend and a great man.

The following is an abstract from the minutes adopted by the Board of Directors of the Pennsylvania Railroad Company at its meeting on March 27, 1929:

"Three years and a half ago, this Board recorded the termination of the active service of Samuel Rea, President of the Company, and paid tribute to his distinguished service. To-day we come, in the sorrow of bereavement, to record the termination of his life,—a life of accomplishment and distinction.

"Though, after his retirement from the Presidency, Mr. Rea continued, as a member of this Board, to devote much of his time, and to give the benefit of his extensive experience and fund of knowledge of its history and traditions, to the management of the Company's affairs, he found, in relief from the exactions of executive office, leisure to devote his talents to the solution of problems of civic welfare, the betterment of his fellow men and the advancement of art.

"Mr. Rea's connection with the Company covered half a century. Innate talent, unusual intellectual endowments, industry, and an indomitable determination to know and to be ever of greater service, developed from the rodman of less than seventeen years the finished engineer and the master of railroad science. Each step in that development was characterized by devotion to the Company's interest and a single purpose to promote its welfare.

"This Company's foothold on Manhattan Island, and the welding of its lines to those in New England, have been called a monument to the vision and courage of Alexander J. Cassatt. The realization of that vision was committed to the hands of Samuel Rea, and its accomplishment stands as the most conspicuous monument to his professional skill and administrative ability. In the corporate and economic structure of the now vast system are other memorials, which, though unseen, are of no less importance to its welfare.

"In recognition of his intellectual attainments, institutions of learning have conferred honorary degrees, and scientific bodies decorations and honorary membership. Worthy tributes all to the man of genius and achievement, but dearer to those who mourn his loss is the willing bestowal of the greatest decoration which man can receive of man—that he was honored, respected and beloved as well for what he was, as for his culture and the fruits of his notable talents.

"The name of Samuel Rea is enrolled on history's pages with those of the distinguished men who have wrought the fabric of, and builded, each in his turn, on the great highway of the nation's progress.

"It is difficult in this record to pay just tribute to the genius and worth of so accomplished a man, and to adequately express the measure of our sorrow and loss in his death.

"His works are his enduring monument; the affectionate regard of those with whom he was associated, a tribute, more eloquent than words, to his worth."

Mr. Rea was elected a Member of the American Society of Civil Engineers on June 4, 1884, and an Honorary Member on June 6, 1921.

EUGENE HILARIAN ABADIE, M. Am. Soc. C. E.*

Died April 27, 1929.

Eugene Hilarian Abadie was born in St. Louis, Mo., on March 1, 1872, the son of Eugene S. and Mary (Snow) Abadie. He was educated in the public and private schools of St. Louis, and at Washington University.

From 1891 to 1892, Mr. Abadie was employed in the Shop and Engineering Department of the Wagner Electric Manufacturing Company assembling and testing electrical apparatus, and from 1893 to 1900 he was Secretary and Manager of Sales, as well as a member of the Executive Board of this Company. From 1901 to 1903, he was Manager of the Sales Organization of the Bullock Electrical Manufacturing Company, of Cincinnati, Ohio, and of the Wagner Electric Manufacturing Company, of St. Louis, directing and supervising the acquisition of engineering contracts.

From 1903 to 1908, he was a member of the firm of E. H. Abadie and Company, Engineers and Contractors, during which time he designed and constructed the heating plant and power house for the State Buildings at Jefferson City, Mo. In 1904, he designed and built the central heating plant for the Pana, Ill., Gas and Electric Company, and, in 1905, he served as Engineer for the Southern Kansas Railway, Light, and Power Company, developed the project, and furnished a report for underwriters.

In 1906, Mr. Abadie designed and built all the underground conduits in Louisville, Ky., for the various public utilities. At that time he was associated with the late Arsene Perrilliat, M. Am. Soc. C. E., in the location of the New Orleans and Baton Rouge Electric Railroad, supervised the engineering, and wrote the engineering report. In 1907, he rehabilitated the Moberly, Mo., Gas and Electric Company and increased the connected load 300 per cent. He also designed and built the municipal power plant at Little Rock, Ark. In 1908, he designed a plant for stripping earth with steam shovels from a coal bed of the Lilly Jellico Coal Company, at Lilly, Ky., and, in a similar manner, mined the coal.

As a Constructing Engineer, Mr. Abadie designed the electric power generating plant for the Consumers Light and Power Company, at Fort Worth, Tex. As a Contractor, he built for the United States War Department, the sewer and water-works at Jefferson Barracks, Missouri. He also designed and installed a number of isolated and industrial steam and electric power plants in St. Louis and in the Central and Southern States. As a general or subcontractor, he contracted to supply mechanical equipment, comprising heating, ventilating, and lighting systems and steam and electrical power plants in shops, department stores, and other large buildings. From 1908 to 1917, he was a practicing Consulting Engineer, and was associated with Dr. George S. Hessenbruch, of St. Louis, during the last five years of this period. In addition, he was Secretary and Treasurer of the Industrial Engineers Corporation of St. Louis. In 1910, he was Consulting Engineer and acted in an advisory capacity to the Public Service Commission in matters connected with the valuation of public utilities of St. Louis.

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* Memoir prepared by D. H. Sawyer, M. Am. Soc. C. E.

In 1917, Mr. Abadie joined the United States Army, first as a Major of Engineers in the United States Reserve Corps. In July, 1917, he was assigned to the Construction Division of the Army as Supervising Constructing Quartermaster, having personal charge of National Army Cantonments and National Guard Camps (Camps Funston, Dodge, and Lee). In September, 1917, Major Abadie took charge of the construction of Camp Lee as Constructing Quartermaster, later going to Washington, D. C., as Supervising Constructing Quartermaster in charge of Camps Holabird, Jessup, and Normoyle, and the Army Tubercular Hospitals, at Denver and Azalea, Colo., as well as several smaller projects. In 1918, he took charge of construction matters connected with shops, camps, etc., of the Motor Transport Corps. In August, 1918, he was commissioned Lieutenant Colonel and served in that capacity until after the World War. In 1919, Colonel Abadie served as General Comptroller for the United States Shipping Board and Emergency Fleet Corporation. On May 1, 1920, he was appointed General Agent, in Washington, for the International Maritime Corporation, Tropical Steamship Corporation, and other shipping concerns. For several years previous to his death, he had been engaged in practice as a Consulting Engineer, with offices in Washington.

Colonel Abadie, due to the wide spread of the engineering field which he served, while maintaining headquarters in St. Louis, acquired an extremely wide circle of friends and business acquaintances. His Army experience and association with the Shipping Board, together with his experience as a Consulting Engineer in Washington, also contributed to his remarkable popularity. He was of a disposition to hold this. He concerned himself in public and organization affairs and was a forceful advocate of policies destined to better his profession, as well as raise the plane of his community.

Colonel Abadie was a member of the American Institute of Electrical Engineers, the American Society of Mechanical Engineers, the Society of American Military Engineers, the Engineers' Club of St. Louis, the Western Society of Engineers, the Society of Colonial Wars, the Military Order of Foreign Wars, the American Legion, the Congressional Country Club, and St. Margaret's Protestant Episcopal Church, in Washington, D. C.

Colonel Abadie was elected a Member of the American Society of Civil Engineers on June 1, 1920.

JOHN JARRETT ALBERTSON, M. Am. Soc. C. E.*

Died November 23, 1928.

John Jarrett Albertson was born in Magnolia, N. J., on August 16, 1858, the son of Chalkley and Annie (Stokes) Albertson. He was descended from old Quaker pioneer stock and the estate on which he lived has been continuously in the Albertson family since 1687, when it was first settled by William Albertson and his wife, Hannah (Druit) Albertson.

John Jarrett Albertson was graduated from the Friends Central High School, at Philadelphia, Pa., in 1876, and for a year, following his graduation, taught school in his home district.

In 1878, he began his life work as an engineer. He was employed by the Philadelphia and Atlantic City Railroad Company on preliminary surveys and, later, on the construction of this railroad to the Jersey Coast. On the completion of this project, he was associated with Judge John Clement, an eminent jurist, and at that time the leading surveyor of South Jersey.

Mr. Albertson took particular interest in road building and was one of the early authorities in New Jersey on Telford, macadam, and gravel roads, writing many of the original specifications governing these types for various State Highway Commissioners. Prior to 1898, he was Engineer for Camden, Gloucester, and Atlantic Counties, New Jersey, engaged on State Aid road work exclusively. At this time, the law required that the County Engineer reside in the county that he represented. Accordingly, Mr. Albertson gave up his work for Atlantic and Gloucester Counties, retaining only his position as Engineer of Camden County, to which he had been appointed in 1892. He held this position until his death.

In 1893, he went to Europe to study road building. After traveling for a year, he returned with much valuable information which he proceeded to utilize in his own work. The location of many of the principal highways of Camden, Atlantic, and Gloucester Counties, remain as they were originally laid out by him, with very minor subsequent deviations.

Probably his foremost contributions to South Jersey were the Meadow Boulevard at Atlantic City, N. J., in the construction of which he introduced hydraulic dredging for road work; the White Horse Pike which has world-wide fame; and the very fine road system of Camden County. He served as Engineer of the Commission in the construction of the Camden County Court House, one of the finest public buildings of its time, and also was influential in the construction of the Camden County Institutions at Lakeland, N. J., where expenditures in recent years have exceeded $4 000 000. He also served with distinction on the Advisory Board for the New Jersey Approaches to the Delaware River Bridge and as a member of the Technical Advisory Committee of the Philadelphia Regional Planning Federation. In addition, he acted as Engineer for many municipalities in Camden County, particularly the Boroughs of Hadden Heights and Audubon which he served from their inception until his death.

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* Memoir prepared by Beale M. Schmucker, Assoc. M. Am. Soc. C. E.

Mr. Albertson was a Mason of high standing, having been a member of Cyrene Commandery and Crescent Temple. He was also a member of the Camden Lodge of the Benevolent and Protective Order of Elks.

He was a man of varied interests. With his brother, he owned and cultivated the homestead farm. He was President and principal owner of the Defiance Fruit Company and also of the Atlantic Cranberry Company, the latter being one of the largest in the State. His home at Magnolia is a horticultural Paradise and here he raised beautiful and rare plants and trees from all climes. His love of horticulture was carried to the County Institutions at Lakeland where former swamp and waste land has been drained and replaced by beautiful shrubbery and flowered gardens.

Mr. Albertson was also active in the line of finance, having been a Director of the First Camden National Bank and Trust Company which is the largest bank in Southern New Jersey and also a Director in the Woodbury Trust Company.

In 1886, he was married to Elizabeth Swift Wills, of Poughkeepsie, N. Y. They had one child, a daughter, Anna Mary, now Mrs. Lester Collins, of Moorestown, N. J.

Mr. Albertson was a forceful man of high character and great energy, whose absolute integrity was never questioned throughout a long and active public life. He was loved and respected by those with whom he came in contact, and his death was a great loss, not only to the community, but to all South Jersey, as he was one of the foremost citizens of the State.

Mr. Albertson was elected a Member of the American Society of Civil Engineers on March 14, 1916.

WILLIAM STUART AUCHINCLOSS, M. Am. Soc. C. E.*

Died April 10, 1928.

William Stuart Auchincloss, the son of John and Elizabeth Auchincloss, was born in New York, N. Y., on March 19, 1842. His primary education was obtained at the Bacon School and at Lyons School, both in New York. Because of his decided mechanical turn of mind he then entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1862, at the head of his class, with the degree of Civil Engineer.

In order to secure the advantage of being well grounded in shop work, Mr. Auchincloss began his career as an employee of the Novelty Iron Works, of New York, where large marine engines were built. At the time, the Company was engaged on the construction of a steamer for the Pacific Mail Route and the iron-clad, *Roanoke,* for the United States Navy.

In 1867, he was appointed by the Government as Commissioner to the Paris Exposition. While abroad on this important errand, he visited Great Britain, France, Germany, Austria, Italy, Egypt, and Palestine. He spent several months at the Exposition and made a complete report to the Government on his return to the United States.

For several years Mr. Auchincloss was connected with the Jersey City Locomotive Works, the Atlantic and Great Western Railway Company, and the Ramapo Car and Foundry Company. After conducting the sale of the Jersey City Locomotive Works, he bought an interest in the Delaware Car Works, of Wilmington, Del., and became Vice-President of the Company.

In 1874, Mr. Auchincloss visited Brazil and built the rolling stock of the São Paulo and Rio de Janeiro Railway. In 1876, subsequent to his return to Wilmington, the Emperor, Don Pedro II, of Brazil visited the United States and was entertained at the home of Mr. and Mrs. Auchincloss. While the Emperor was in this country he ordered a private car built by the Delaware Car Works, which was designed and built under the supervision of Mr. Auchincloss. This car was exhibited at the Philadelphia Centennial Exposition in 1876 and received first prize.

Mr. Auchincloss moved from Wilmington to Philadelphia, Pa., in 1879, and organized the firm of Bates and Auchincloss, Agents of Messrs. J. and P. Coats, Thread Manufacturers, of Paisley, Scotland. After Mr. Bates died, Mr. Auchincloss carried on the business in his own name, establishing his family, and building a home, in Bryn Mawr, Pa., which he called "Hillcrest".

In 1903, having retired from business, he moved to Atlantic Highlands, N. J., where he built another home, "Rest Haven", on the shore of Sandy Hook Bay. Here, he lived until his death on April 10, 1928.

Mr. Auchincloss' tastes were decidedly mechanical, but, in later years, he devoted much of his time to literary work. He was the author of the following books and booklets: "Report to U. S. Government, as Honorary Commissioner to the Paris Exposition" (1867); "Standard Work on Link and Valve

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* Memoir prepared by William K. Auchincloss, Esq., New York, N. Y.

Motions" (1869); "90 Days in the Tropics" (1874); "Schieber und Contissendteurungen" ("Link and Valve Motions", published in Germany, 1886); "Yearly Tides" (1892); "Waters within the Earth and Laws of Rainflow" (1897); "Stoomschief-en Schaarbewegingen" ("Link and Valve Motions", published in Holland, 1899); "St. Peter, the Apostle of Asia" (1901); "The Book of Daniel Unlocked" (1905); "The Mask of Eddyism" (1907); "Standard Chronology of the Holy Bible" (1908); "Wonders of God's Universe" (1912); "Seaside Astronomy" (1920); "Christianity and the Britons" (1920); "The Greatest Man of the Ages" (1921); and the final edition of the "Standard Chronology of the Holy Bible" (1924).

Mr. Auchincloss also directed his genius to the invention of an averaging machine in 1882. This machine had a capacity of 110 business accounts per hour, which equalled the customary task of six skilled accountants. A paper describing the machine, entitled "Averaging Machine",* was presented by its inventor before the Society. Relative to his mechanical work, Mr. Auchincloss had received six patents.

In 1870, he was married to Martha Tuthill Kent, the daughter of William Campfield Kent, a noted merchant of the firm of James Kent Santee and Company, of Philadelphia. They had three children, James Stuart, Jane Kent, and William Kent.

He was most kindly, upright, and honorable in all his dealings—a devoted father, and a Christian gentleman.

Mr. Auchincloss was elected a Member of the American Society of Civil Engineers on February 7, 1869.

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* *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 121.

WILLIAM AUSTIN BASSETT, M. Am. Soc. C. E.*

DIED MAY 16, 1929.

William Austin Bassett was born on September 29, 1876, at Roxbury, Mass. His New England ancestry dates back to 1621. His father was Isaac Austin Bassett and his mother, Annie (Tuson) Bassett.

Mr. Bassett attended the Lawrence Scientific School at Harvard University, from which he was graduated in 1901, with the degree of Civil Engineer. From August, 1901, to October, 1904, he was with the Pittsburgh Division of the Pennsylvania Railroad Company, in the Altoona, Pa., Office, in charge of construction of two main-line bridges, double-tracking the Southwest Branch, and miscellaneous yard and terminal work. From October, 1904, to June, 1906, he was with the Pittsburgh, Pa., Bureau of Filtration, engaged in the construction of a filtration plant, filtration galleries, and settling basins of reinforced concrete design. In June, 1906, he was appointed Assistant Professor of Civil Engineering at Carnegie Technical School, at Pittsburgh, and also was engaged in special work for the Department of Water Supply of the City of Pittsburgh.

From June to November, 1909, Professor Bassett was with the New York City Board of Water Supply as Assistant Engineer on the construction of the Rondout Tunnel at High Falls, N. Y., and from November, 1909, to November, 1910, he was Designing Engineer in charge of the drafting-room at the Cambridge Office of the Boston Elevated Railroad Company, engaged in the construction of the Cambridge-Boston Subway. From November, 1910, to November, 1911, he was Engineer in charge of construction, costs, and cost investigations, at the Donora, Pa., Steel Works, of the American Steel and Wire Company. From November, 1911, to April, 1913, he served as Associate Editor of *The Engineering Record.*

From April, 1913 to December, 1926, Professor Bassett was Chief of the Division of Engineering and Public Works of the New York Bureau of Municipal Research and National Institute of Public Administration, engaged in miscellaneous professional and consultant service to municipalities, counties, and States in the United States and Canada. On December 1, 1926, he joined the Faculty of the Massachusetts Institute of Technology to organize a Division of Municipal and Industrial Research, of which he was Director until his death. Some of the accomplishments under his direction were a comprehensive survey of industrial conditions and possibilities in and about Providence, R. I., an industrial survey of Bangor, Me., and civic surveys of Meriden, Conn., and Norwood, Mass.

He served on the Hoover Committee for the Simplification of Building Codes and Practices, and on the Advisory Board for Highway Research of the National Research Council. He was a Trustee of the Village of Scarsdale, N. Y., from 1922 to 1926, and a Supervisor of Westchester County, New York, from 1923 to 1926. He was the author of "Problems of Road Administration", published in 1917, and contributed to the technical press on engineering matters related to municipal and State problems.

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* Memoir prepared by William W. Brush, M. Am. Soc. C. E.

Professor Bassett was intensely interested in establishing higher and more efficient standards in carrying on municipal and other Governmental activities, and devoted a great deal of his time to this phase of engineering work. His criticisms were always constructive and the methods he adopted, together with his pleasant personality, added greatly in securing the co-operation of those with whom he came in contact. His death at a comparatively early age is a distinct loss to the community.

He was a member of the Harvard Society of Engineers, the American Society for Municipal Improvements, and the Harvard Club.

His widow, Grace Loring Bassett, to whom he was married on March 30, 1908, and a son, John Austin Bassett, survive him.

Professor Bassett was elected a Member of the American Society of Civil Engineers on August 9, 1920.

SAMUEL GIVENS BENNETT, M. Am. Soc. C. E.*

DIED APRIL 4, 1929.

Samuel Givens Bennett, the son of John W. and Agnes (Givens) Bennett, was born at Taylorsville, Ky., on July 12, 1863. He was educated in the public schools at Owensboro and Hartford, Ky., and later studied for two and one-half years at Georgetown College, Georgetown, Ky.

In 1886, Mr. Bennett went to California where he commenced his life work, the practice of land surveying and civil engineering. His first employment was with the late E. T. Wright, M. Am. Soc. C. E., of Los Angeles, Calif., in the capacity of Chainman, Rodman, and Draftsman.

In subsequent years he was engaged in the following positions: From September, 1889, to September, 1890, as Draftsman and Instrumentman with L. Friel, Civil Engineer; from September, 1890, to September, 1891, as Assistant Engineer with the Redondo Railway Company; from September, 1891, to October, 1893, as Draftsman with the Pacific Coast Abstract Company, of Los Angeles, Calif.; from October, 1893, to December, 1896, in conducting general engineering business in Los Angeles and vicinity, in partnership with Frank H. Olmsted, M. Am. Soc. C. E., and the late Burr Bassell, M. Am. Soc. C. E.; from May, 1897, to April, 1898, as Engineer with the Chicala Water Company, of Rialto, Calif.; from April to October, 1898, as Superintendent of the construction of a pumping plant and a riveted steel pipe line for the Azusa Glendora Water Company; from October, 1898, to April, 1899, in association with J. B. Lippincott, M. Am. Soc. C. E., in connection with the Pasadena, Calif., Water Supply System; from April, 1899, to January, 1903 as Assistant Resident Hydrographer for the United States Geological Survey in California; from January, 1903, to June, 1906, as Engineer in the United States Reclamation Service, having charge of the Sacramento Valley Project for almost two years of this period; in 1907, as Locating Engineer for the Los Angeles and Owens Valley Railroad Company; from 1908 to 1909, in private practice; and during 1910, as Engineer for the United Sugar Company in the State of Sinaloa, Mexico. In 1911, Mr. Bennett again returned to private practice, but was soon appointed City Engineer of Oxnard, Calif., which position he held until his death.

On March 22, 1899, Mr. Bennett was married at Redlands, Calif., to Mable J. Randall, of Minneapolis, Minn., who, with their four children, Harman R., S. Gerald, Mable Rose, and John Newton, survives him. Mr. Bennett was a member of the Baptist Church, and was greatly beloved among a wide circle of friends.

Of a retiring, modest disposition, he was a great student along scientific and religious lines. His hydrographic work in the High Sierras in California for the U. S. Geological Survey brought him conspicuous success. As City Engineer of Oxnard, "Sam" Bennett enjoyed the esteem and respect of the Trustees of that city during the long period of years in which he held that office.

Mr. Bennett was elected a Member of the American Society of Civil Engineers on October 3, 1906.

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* Memoir prepared by Frank H. Olmsted, M. Am. Soc. C. E.

EDWARD AUSTIN BOND, M. Am. Soc. C. E.*

DIED DECEMBER 10, 1929.

Edward Austin Bond was born on April 22, 1849, at Dexter, Mich., the son of Hollis and Emily Faxon Bond. His ancestor, William Bond, came to the United States from England about 1620 and settled at Watertown, Mass.

Mr. Bond was educated in the public schools of Michigan and in a business college at Utica, N. Y. His first engineering work was for the Utica, Chenango and Susquehanna Railroad Company in 1868. In 1869, he served as Assistant Engineer on the Chicago, Pekin, and South Western Railroad, near Pekin, Ill., and on the Utica, Chenango and Susquehanna Valley Railroad, near Norwich, N. Y.

In 1870, he became Assistant Engineer on the Utica and Black River Railroad from Lowville to Carthage, N. Y., and also on the Black River and Morristown Railroad, near Philadelphia, N. Y. From late in 1870 to November, 1873, he was Chief Engineer of the Clayton and Theresa Railroad, from Clayton to Theresa Junction, N. Y. During part of 1874, Mr. Bond served as Resident Engineer, and between August, 1874, and April, 1886, as Chief Engineer, of the Utica and Black River Railroad and during that time the line between Morristown and Ogdensburg, N. Y., was built. From May, 1886, to May 1, 1889, he was Chief Engineer and General Superintendent of the Carthage and Adirondack Railroad, on the portion between Carthage and Benson Mines, N. Y. Several railroads on which Mr. Bond was employed are now included in the New York Central Lines.

Mr. Bond was a member of the engineering firm of Hinds and Bond, of Watertown, N. Y., from 1889 to 1896, and during those years this firm designed or constructed water-works in Cape Vincent, Antwerp, Theresa, West Carthage, and Philadelphia, N. Y., and Barrie, Chatham, and Napanee, Ont., Canada. Mr. Bond had personal charge of the location of the Gouverneur and Edwards Railroad and of the surveys for the proposed Mohawk and St. Lawrence Railroad in Jefferson, Lewis, and St. Lawrence Counties, New York.

He was engaged in private practice from July, 1896, to November, 1898, as Consulting Engineer on various works. In November, 1898, he was elected State Engineer for New York and served as such until May 1, 1904. While Mr. Bond held this position many questions on canal and highway matters were considered. He served on the Committee on Canals that thoroughly investigated different projects in this field and reported to the Governor thereon in January, 1900. As State Engineer he had charge of a detailed survey of the canal routes through the State, known as the Barge Canal Survey, and reported thereon to the Governor early in 1901. The report gives in detail the various canal routes, estimates of cost, reports of the engineers in charge of the various parts of the work, and, in all, is a record of great engineering value. The studies covered by the report were so thorough and the methods used so exact that they received the confidence of the people of the State and enabled them to form a careful judgment on the matters considered. As State Engineer, Mr. Bond also had charge of the State's improved roads—then in their infancy—and many of his designs have since come into general use.

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* Memoir prepared by William B. Landreth, M. Am. Soc. C. E.

Mr. Bond resigned as State Engineer on May 1, 1904, to become Chairman of the Advisory Board of Consulting Engineers on the Barge Canal, and served as such until that Board was abolished by the State Legislature, in July, 1911. While the duties of the Advisory Board varied somewhat from time to time, it had a general oversight of the design and construction of the Barge Canal. Mr. Bond visited Europe as a member of the Barge Canal Terminal Commission that made an exhaustive study of terminals and, in the main, the report of that Commission has been followed on State Terminals.

After 1911, Mr. Bond was in private practice on canal and harbor works. He visited Uruguay on a harbor project in 1914, and was a member of the Advisory Board of the Trans-Alaska and Siberia Railroad.

He had the faculty of choosing competent and experienced men to work with him, and he showed much tact and wisdom in such choice. He quickly realized the essential points of a project and his decisions showed a full knowledge of the questions involved and their solution. He was a good judge of men and their work, and his associates showed their confidence in him by giving of their best efforts.

Mr. Bond was a Thirty-third Degree Mason, and a Republican in his political affiliations. He was an officer in several water-works companies in the United States and Canada.

He was married to Gertrude Hollenbeck in 1873; after her death to Clara E. Ellis, in 1904; and subsequent to her death to Mrs. Elizabeth Parsons, in 1908, who survives him.

Mr. Bond was elected a Member of the American Society of Civil Engineers on September 6, 1899.

WILLIAM LEWIS BRECKINRIDGE, M. Am. Soc. C. E.*

Died July 11, 1929.

William Lewis Breckinridge, the son of Marcus Prevost Breckinridge, M. D., and Lucy (Long) Breckinridge, was born at Louisville, Ky., on June 29, 1857.

He was a direct descendant on his paternal side of a family noted for its patriotism, oratory, and statesmanship. After first settling in Virginia in 1730, John Breckinridge, his great-grandfather, migrated to Kentucky in 1780. One of his grandsons, John C. Breckinridge, became famous in political circles, being Vice-President of the United States from 1856 to 1860, and, later, running for President of the United States in opposition to Abraham Lincoln. He afterward became a General in the Confederacy and Secretary of War under Jefferson Davis.

His grandfather, the Rev. William Lewis Breckinridge II, was a Presbyterian Minister, and, at one time, served as Moderator of the combined Presbyterian Churches of the United States. While many of the Breckinridge family espoused the Southern cause, Marcus Prevost Breckinridge and his descendants allied themselves with the Union in the controversies leading up to the Civil War.

On his maternal side, the Long family was quite as prominent; his grandfather, Stephen Harriman Long, was one of the foremost engineers of his day. From 1818 to 1826, he supervised explorations between the Mississippi River and the Rocky Mountains, Long's Peak, the highest summit in those mountains, being named after him. In 1829, he published a "Railroad Manual", the first book of its kind in the United States. From 1827 to 1830, he was Chief Engineer of Surveys, in connection with the building of the Baltimore and Ohio Railroad; later, he became Engineer in Chief of the Western and Atlantic Railroad, in Georgia, during which time he introduced a system of curves on location work. He served for a time on the Board for Improvements of the Mississippi River, and was made Colonel of Engineers in 1861, shortly before his retirement in 1863.

William Lewis Breckinridge spent his boyhood days in Alton, Ill., attending the public schools. After graduating from High School, he went to the Worcester Military Academy, Worcester, Mass., and from there to Washington University, St. Louis, Mo., from which he was graduated with the degree of Civil Engineer in 1879. In June, 1907, Washington University conferred upon him the Honorary Degree of Master of Arts.

He entered the service of the Chicago, Burlington, and Quincy Railroad Company on July 15, 1879. Beginning as an Assistant Engineer, he was subsequently made a Division Engineer in 1880; Engineer of Iowa Lines in 1890; Assistant Superintendent of Iowa Lines in 1898; Chief Engineer in 1899; Chief Engineer of Maintenance in 1904; and Chief Engineer of the Chicago, Burlington, and Quincy System, then under Federal Control, in 1918. He continued in this capacity until the end of Federal Control in 1920, when he was appointed Assistant Chief Engineer of the System, which position he held until his retirement in April, 1923.

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* Memoir prepared by a Joint Committee of the Society, the Western Society of Engineers, and the Chicago Engineers Club, consisting of A. W. Newton and Elmer T. Howson, Members, Am. Soc. C. E., and Charles W. Melcher, M. W. S. E.

In his service with the Burlington Railroad, extending over a period of nearly half a century, Mr. Breckinridge saw the Lines East of the Missouri River grow by construction and by acquisition from a group of loosely knit light traffic lines, comprising a total of 1 857 miles in 1880, to a highly developed system of 4 410 miles in 1923. This period was one of intensive, as well as extensive, development, much of which was under Mr. Breckinridge's supervision. He was in charge of the re-alignment and revision of grades on the Burlington's main line across Iowa from 1898 to 1902, which comprised some of the heaviest work of its kind up to that time.

Under his administration, the Burlington extended its line into the Southern Illinois coal fields, and then rebuilt existing lines from Centralia, Ill., north to St. Paul, Minn., providing a maximum grade of 0.3% for this distance of more than 600 miles. Coincident with this program, numerous large terminals were built, including the gravity classification yards at Galesburg and Hawthorne, Ill., and the new locomotive shops at West Burlington. Particularly outstanding for its day was the track-elevation work between Chicago and Clyde, Ill., one of the earliest projects of this magnitude to be undertaken.

Mr. Breckinridge was a man of unusual personality, possessing a dignity of bearing, yet with a friendliness that inspired respect and admiration. He was a gentleman in every sense of the word, considerate of others to an unusual degree—a trait which won for him the unreserved loyalty and friendship of all those with whom he came in contact. Naturally of a retiring disposition, his ability and personality were known only to those with whom he was associated, but by them he was held in the highest esteem.

On October 15, 1891, he was married to Irene Waples, of Alton, Ill., who, with two sons, William Lewis and Frank Prevost, survives him. His religious affiliations were with the Protestant Episcopal Church.

He became a member of the Western Society of Engineers and of the American Railway Engineering Association in 1899, and of the Chicago Engineers' Club in 1906.

Mr. Breckinridge was elected a Member of the American Society of Civil Engineers on October 7, 1903.

WILLIAM WALLACE BRIER, M. Am. Soc. C. E.*

Died December 28, 1928.

William Wallace Brier, the son of the Rev. William Wallace Brier, a Presbyterian home missionary, and Elizabeth Ann (Naylor) Brier, was born near Centerville, Calif., on August 7, 1858. He was a great-grandson of Lieutenant Benjamin Lodge, of the Pennsylvania Continental Line, one of the original members of the Society of the Cincinnati. His father was of Scotch-English-Quaker ancestry, some of his forebears having settled in Pennsylvania after buying large tracts of land from William Penn in 1682. His mother was of Irish-English descent.

Mr. Brier attended the public school near his father's farm, and, under his tutorship, prepared for the University of California, from which he was graduated in 1882 with the Degree of Bachelor of Philosophy in Civil Engineering.

In 1882 and 1883, Mr. Brier was employed on maintenance of way and water supply work as an Assistant to the Division Engineer of the Galveston, Harrisburg and San Antonio Railway Company, a subsidiary of the Southern Pacific Railroad.

From 1884 to 1890, he taught school in San Luis Obispo and Los Angeles Counties, California, and conducted a book business in San Francisco, Calif. He returned to engineering in 1891, and was in private practice until 1897, in which year he was appointed Civil Engineer of the Alameda Sugar Company, at Alvarado, Calif., and, later, of the Union Sugar Company, at Betteravia, Calif., in charge of railway, irrigation, drainage, and reclamation work.

Mr. Brier remained in these positions until March, 1901, at which time he became Chief Engineer of the E. B. and A. L. Stone Company, General Contractors, at San Francisco. During his connection with this firm, which continued until the end of 1905, he superintended the construction of the plant of the Standard Portland Cement Company at Napa Junction, Calif., the erection of the Los Angeles and Salt Lake Railroad Company's concrete bridge across the Santa Ana River, in California, and also had charge of numerous other contracts which the firm was executing.

While he was superintending construction, Mr. Brier had made a study of contractors' methods and, after leaving the Stone Company, he decided to make use of his acquired experience. In March, 1906, he organized the Occidental Construction Company, of Los Angeles, Calif., which specialized in railroad construction, and of which he was President and Engineer. In 1917, he founded the Western Reinforced Concrete Pipe Company, of Los Angeles, which was engaged in the design, manufacture, and installation of this class of pipe, and of which he was also President and Engineer. Early in 1923, he disposed of his interests in Los Angeles and moved to San Francisco, where he maintained an office as a Consulting Engineer until a few months before his death.

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* Memoir prepared by the following Committee of the San Francisco Section: Bernard Benfield and Jerome Newman, Members, Am. Soc. C. E.

Mr. Brier was married on March 27, 1889, to Helen Maria Blake who, with four children, William Wallace III, Edward Blake, Helen Naylor, and Isabel Presbrey Brier, M. D., and two sisters, Mrs. Mary Brier Moores and Mrs. Carrie Brier Schorr, survives him.

In all the relations of life, as a dutiful son, husband, and father, as a friend, engineer, contractor, and citizen, William Wallace Brier was a modest, kindly, sterling character. Firm in his beliefs, yet wholly tolerant and considerate of the views of others; loyal to those whom he served and appreciative of those who served under him; public-spirited, and deeply interested in all matters affecting the Commonwealth; faithful to every promise and every obligation; he gave freely and efficiently of the best that was in him in everything that he undertook. The world can ill afford to lose a man such as he was; his passing left a deep sense of loss in the hearts and memories of those who were privileged to be associated with him in any phase of his long and active career.

Mr. Brier was a member of the Commonwealth Club of California, the Engineers Club of San Francisco, the Astronomical Society of the Pacific, the Seismological Society of America, the American Association of Engineers, and the American Association for the Advancement of Science. He was also a member of the Masonic Order and of Phi Gamma Delta Fraternity.

Mr. Brier was elected a Member of the American Society of Civil Engineers on October 21, 1924.

WALDO CLAYTON BRIGGS, M. Am. Soc. C. E.*

Died December 13, 1926.

Waldo Clayton Briggs was born in New Haven, Conn., on March 14, 1870. He was the son of the late William Alexander Briggs, a merchant of New Haven, and Sarah Maria (Baldwin) Briggs, both of whom traced their ancestry back to the Colonial days of New England. His forebears fought on the patriot side in the American Revolution.

Mr. Briggs attended the public schools of New Haven and was graduated from the Hillhouse High School in 1889. The following three years he spent in the Sheffield Scientific School of Yale University, from which he was graduated in 1892 with the degree of Bachelor of Philosophy.

During his vacation periods, while he was in college, Mr. Briggs worked on surveying and drafting for the City Engineer of New Haven, and, in 1892, following his graduation, he continued in the same type of work with the City Engineer of Hartford, Conn. In 1893, he returned to New Haven as Resident Engineer on the construction of a masonry dam, about 1 000 ft. long, for the New Haven Water Company.

From 1894 to 1897, he served as City Engineer and Street Commissioner for South Norwalk, Conn. His next engagement, covering the period from 1898 to 1900, was as a Civilian Engineer with the Corps of Engineers, United States Army, on computing, drafting, and superintending the construction of fortifications at the eastern entrance to Long Island Sound. This connection was severed when the appropriation for the work was exhausted.

Mr. Briggs went to New York City in 1900 and, thereafter, until the time of his death, he was engaged on the construction of important engineering projects in and about the city. From October, 1900, to March, 1901, when he resigned, he was an Assistant Engineer on the First Division, Rapid Transit Railroad Commissioners, supervising the construction of a part of the first rapid transit subway. Subsequently, until 1904, he was employed with the Degnon-McLean Contracting Company, as Assistant Engineer, later as Engineer in Charge, under the Chief Engineer of the Company, on its contract for the building of that part of the first rapid transit subway which extended from 41st Street and Park Avenue, through 42d Street and Broadway, to 47th Street. At that time, subway construction was of the nature of pioneer work in New York City, and experience in working out the problems involved, by which those who built the later subways benefited, was then lacking. Those, like Mr. Briggs, who so successfully accomplished their difficult task under such conditions, deserve commendation for the ingenuity, foresight, and skill they displayed.

In 1905, when the Pennsylvania Railroad Company began its construction of the great project for a passenger terminal in New York City with all the appurtenant work, including tunnels under the Hudson and East Rivers, Mr. Briggs became the Resident Engineer for the New York Contracting and Trucking Company on its contract for excavating the site of the passenger terminal in the blocks bounded by 31st and 33d Streets and Seventh and Ninth Avenues, and for constructing the retaining walls and viaducts in connection with this project.

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* Memoir prepared by Robert Ridgway, Past-President, Am. Soc. C. E.

In 1907, he accepted a position with the Degnon Contracting Company, and, at first, was engaged on the preparation of bids and the supervision of various contracts. The following year he was made Chief Engineer of the Degnon Realty and Terminal Improvement Company, and was in charge of the contract for the construction of the large Sunnyside Yards of the Pennsylvania Railroad and of the real estate development in Long Island City, N. Y., continuing in this capacity to the end of 1912. The successful completion of the construction of the Sunnyside Yards, in low-lying marsh land, gave him much satisfaction.

From 1913 to 1916, Mr. Briggs was Chief Engineer for Carpenter and Boxley and Herrick on the construction of Sections 11-B-1 and 11-B-2 of the rapid transit subway from 40th Street to 86th Street, in Fourth Avenue, Brooklyn, N. Y., at a cost of $3 725 000. On the completion of this work, he returned to the Degnon Contracting Company, for the period, 1917 to 1920, as Engineer and Superintendent on two other Brooklyn subway contracts, one a deep tunnel in sand, 4 300 ft. long, known as Section 2-A of Route 12, the other, a subway station, directly under the existing Flatbush Avenue Terminal of the Long Island Railroad, known as Section 1-B of Route 12. These contracts involved a cost of about $2 000 000. Although Section 1-B was of relatively short length, the construction was so involved as to require careful planning and execution. Mr. Briggs brought both contracts to successful completion, after which he severed his connection with the Degnon Contracting Company, as that Company had decided to discontinue its contracting activities.

During 1922, the Thomas Crimmins Contracting Company of New York City employed Mr. Briggs as Superintendent on a $2 000 000 contract in Westchester County for installing additional steel pipes in certain siphons on the Catskill Aqueduct of the New York City Water Supply. From 1923 to the time of his death in 1926 he was employed by Henry Steers, Incorporated, of New York City, as Engineer and Superintendent. For the major part of these four years he was engaged on the difficult task of installing foundations for the large plant of the Western Electric Company at Kearney, N. J., on the marshy ground of the Passaic River meadows. The contract involved the expenditure of about $4 000 000, and, in the words of a member of Henry Steers, Incorporated:

> "Not only by his engineering ability as shown by the efficient and able handling of this intricate work, but by his experience, his personality, and the many sterling qualities of his character, which we came to know, he established himself as a permanent and valued member of our staff. He acted in an advisory capacity to Mr. Steers on the various projects in which we were interested, and at the time of his death, he had an assured and brilliant future in this Company. By his untimely end, we feel deeply the loss of a valued business associate and a friend."

Death came to Mr. Briggs suddenly, without warning. On the morning of December 13, 1926, while leaving the City Hall of Jersey City, N. J., where he had gone on business connected with plans for work in progress in that city, he had a heart attack and died almost instantly.

Waldo Briggs was a man of positive characteristics—one to whom right was right and wrong was wrong. He was intolerant of anything that was not direct and open and drove straight to the conclusion that, in his judgment, was the correct one. Absolutely loyal to a friend and a cause, he was indefatigable in his attention to any work he had in hand, studying its particular problems, never satisfied until he had mastered its details and thought out ways of overcoming its difficulties. His success was due, in large measure, to his straightforward and courageous way of looking at his problems and to his courage in acting as his judgment directed. One remembers long, and appreciates deeply, the friendship of such a man. He was of the army of constructive men who leave the world better than they found it.

Mr. Briggs enjoyed country life and out-of-doors sports. He was fond of dogs and on his long rambles, one, and sometimes two, were his companions. Biographies of noted men always interested him, and he read many, making copious notes of his impressions. In line with this, his interest in civic affairs was deep and he took the active part of a good citizen in the local affairs of his community.

He was married on September 2, 1897, to Belle Ferris, of South Norwalk, Conn., who, with a son, Ferris Briggs and a daughter, Amy Ferris Briggs, survives him. He was a member of the Scarsdale Golf Club, the Carteret Club of Jersey City, and the Yale and Quill Clubs of New York City.

Mr. Briggs was elected a Junior of the American Society of Civil Engineers on December 5, 1893, and a Member on July 10, 1907.

ALEXANDER BROCINER, M. Am. Soc. C. E.*

Died November 16, 1928.

Alexander Brociner was born at Galatz, Roumania, on April 21, 1880, the son of André and Rosalie Brociner. His early life was spent in his native city, where he obtained his elementary and higher education, receiving his Bachelor's degree in 1898. Five years later the Royal Technical College at Bucharest, Roumania, from which he was graduated with highest honors, conferred on him the degree of Civil Engineer. He then served for a time as Second Lieutenant of Heavy Artillery in the Roumanian Army.

While he was still a student, Mr. Brociner had visualized the United States as the country offering the widest scope for his activities. Accordingly, shortly after his graduation, he came to America.

His practical engineering experience began with the firm of Bradford Lee Gilbert, in New York, N. Y., for which he soon became Chief Engineer, serving in this capacity from 1904 to 1907. At this time Mr. Brociner began his independent practice as Consulting Engineer, on building and heavy construction. He was soon rewarded with success. For more than twenty years, he had an extensive practice, during which he greatly increased the scope of his work, having to his credit apartment houses, hotels, loft buildings, churches, school buildings, railroad stations, water-front structures, bridges, etc. Among the buildings for which he was Consulting Engineer are the Windsor Hotel, Montreal, Que., Canada, the Chateau Laurier and the Terminal Station, Ottawa, Ont., Canada, Lawrence College, Yonkers, N. Y., the King Cotton Hotel, Greensboro, N. C., and the Proctor Memorial Bridge, at Proctor, Vt.

At various times Mr. Brociner contributed technical articles to *Engineering News-Record, Insurance Press, Safety Engineering,* and its predecessor, *Insurance Engineering,* the *Real Estate Magazine,* and also to the New York daily press.

He was an untiring worker who never spared himself when his time and energy were needed. His work was especially marked by a thorough conscientiousness in the interest of his clients. In his particular field of structural engineering, he was a close student of new developments, and his thorough knowledge of good practice and legal requirements made his services of great value. The high regard in which he was held by his profession is expressed in a memorial resolution of the Structural Engineers' Society of New York, of which he was a member, in the following terms, as

"An engineer of ability and highly respected both for his accomplishments and also for his high standard of professional ethics.

"He deserves high credit for his labors in the Society and in an effort to create a better understanding and bond of fellowship among his co-workers."

He is survived by his widow, Jolanthe, to whom he was married in 1909, and by a son, Victor.

Mr. Brociner was elected a Member of the American Society of Civil Engineers on August 28, 1922.

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* Memoir prepared by Jacques S. Negru, Esq., New York, N. Y., and Rudolph P. Miller, M. Am. Soc. C. E.

WILLIAM BROWN, M. Am. Soc. C. E.*

Died December 24, 1929.

William Brown was born in Glasgow, Scotland, in 1850, and was educated at Renfrew and Paisley Grammar Schools and at the University of his native town. He was the son of Andrew Brown, who invented and patented the type of vessel now known as a "hopper dredge". This design was improved in later years by the introduction of the "stern well type" which also was patented by Mr. Andrew Brown.

Beginning his apprenticeship in 1866 in the Engineering Department of Messrs. William Simons and Company of Renfrew, William Brown was assumed with his brother, Col. Walter Brown, as a partner by his father. He was associated with this firm, which was converted into a Limited Liability Company in 1900, until his death on December 24, 1929.

Mr. Brown was appointed Assistant Manager of the Engineering Department in 1872 and Manager in 1876. He was taken into partnership in 1888, and was elected a Director in the Company when it was floated. He then became Chairman and Managing Director in 1907, which position he held until his death.

He identified himself very closely with the development of all types of marine dredging plant, particularly suction dredges and in the adaptation of this type of dredge for working in clay or similar materials. He patented a number of devices calculated to increase the efficiency of such machinery. The large fleet of dredging plant originally used at Durban, Natal, for the improvement of the depths in the bar and harbor there, and a large number of very powerful dredges which are in use at various ports in India, were designed under his direction.

Mr. Brown was a Member of the Institution of Civil Engineers, the Institution of Naval Architects, the Institution of Mechanical Engineers, and the Institution of Engineers and Shipbuilders in Scotland. At various times, he contributed to the *Proceedings* of the first and last named. He also found time to devote himself very largely to the public interests of Renfrew, and he represented the Renfrew Town Council on the Clyde Navigation Trust for a great number of years.

In 1918 Mr. Brown had the honor of Commander of the British Empire conferred on him in recognition of his services during the World War; and, in 1923, the Burgh of Renfrew further honored him by presenting him with the Freedom of the Burgh.

Mr. Brown was elected a Member of the American Society of Civil Engineers on October 3, 1911.

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* Memoir prepared by Wm. Simons & Co., Ltd., Glasgow, Scotland.

RALPH ELIJAH BROWNELL, M. Am. Soc. C. E.*

Died January 29, 1928.

Ralph Elijah Brownell was born in Filmore County, Minnesota, on January 5, 1857. He received his primary education in the public schools, and was graduated from the Englewood, Ill., High School in June, 1874.

Mr. Brownell then entered the Department of Public Works of the Town of Lake (now a part of Chicago), Ill., with which he was connected in every branch of municipal work until January, 1880. During four years of this period he received instruction in Mathematics and Civil Engineering under the private tutelage of the Principal of the Englewood High School. From January, 1880, to January, 1882, Mr. Brownell was Assistant Engineer with the South Park Commission of Chicago and from January, 1882, to March, 1883, he held a similar position with the Grand Trunk Railway Company on reconstruction work throughout Michigan, which was followed by his acceptance of the position of Chief Engineer and Superintendent of Public Works of the Town of Lake.

In March, 1886, Mr. Brownell became a member of the firm of Eggleston, Mallette, and Brownell, which firm was engaged in general contracting for public improvements. He was the responsible head of this organization in the development of property in the suburbs of Chicago and in this capacity had charge of all engineering work and its various resultant problems. In September, 1894, he organized the Brownell Improvement Company of Chicago, of which he was, successively, Vice-President, President, and again, Vice-President. This Company was engaged in subdividing and improving vacant property in the suburbs of Chicago. In January, 1902, he was made Consulting Engineer for the City of Chicago, and in this position was engaged particularly in the design of stone-crushing plants and in public works.

In June, 1904, Mr. Brownell moved to Oklahoma City, Okla., where he organized and operated the O. K. Building and Construction Company, Contractors, engaged in public improvements, principally in concrete construction. From November, 1907, to May, 1909, he served as Consulting Engineer for Oklahoma City; from May, 1909, to May, 1911, as Chief Engineer of the Park Department, designing the Park System and constructing a 28-mile boulevard around the city; and from May, 1911, to November, 1914, he again served as Consulting Engineer for Oklahoma City. In November, 1914, he was appointed County Engineer for Greer, McIntosh, and Pottawatomie Counties in the State of Oklahoma, which position he retained until August, 1916, when he became District Engineer of the Portland Cement Association for Oklahoma, and was thus engaged until August, 1922. In this connection, Mr. Brownell was primarily instrumental in promoting the building of concrete roads throughout the State.

Mr. Brownell first went to Los Angeles, Calif., in 1922, with the Rotary Club of Oklahoma City, and immediately became interested in the development of The Palos Verdes Estates, near Redondo Beach, Calif., planning and building the water-works for that organization. His death occurred while he was actively engaged on this construction.

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* Memoir prepared from information on file at the Headquarters of the Society.

He was a member of the American Association of Engineers, having served as President of the Oklahoma City Chapter. He was also a member of the Redondo Beach Rotary Club, and a Trustee of the Methodist Episcopal Church at that place, actively associated in its Sunday School work.

A man of impeccable character and of true fineness of nature, Mr. Brownell was loved and respected by all those of the Engineering Profession by whom or with whom he had been employed or associated.

On October 31, 1878, Mr. Brownell was married to Lucy T. Adams, who, with four daughters, survives him.

Mr. Brownell was elected a Member of the American Society of Civil Engineers on November 9, 1920.

STEPHEN SANS BUNKER, M. Am. Soc. C. E.*

Died December 15, 1928.

Stephen Sans Bunker, the son of David A. and Jeannette (Higgins) Bunker, was born at Bar Harbor, Me., on August 20, 1876. His education was obtained in the Bar Harbor schools and at the University of Maine in Orono. He was graduated from the latter institution in the Class of 1897, receiving the degree of Civil Engineer. He was a Member of Phi Gamma Delta Fraternity.

Immediately on the completion of his college course Mr. Bunker went to Rockland, Me., where for several months he held a position in the City Engineer's Department. He then accepted a position as Transitman on railroad construction with the James T. McDonald Company of New York, N. Y., remaining with that firm for three years. During this period he was employed in the construction of the Washington County Railroad, now a branch of the Maine Central Lines, and on the Seaboard Air Line in Virginia.

In 1900, he returned to Bar Harbor, where he opened an engineering office and was actively engaged in building an addition to the water-works of the town. From 1901 to 1902 he was employed with the W. G. Wilkins Engineering Company, of Pittsburgh, Pa., holding the position of Resident Engineer in street railway and paving construction. In the fall of 1902 he became connected with the Jersey City, N. J., Water Supply Company in the building of a 6-ft., steel, water main for that city. From 1902 to 1904 he was Chief of Party and Resident Engineer on railroad construction in Oklahoma with the William Kenefich Construction Company.

In 1904 and 1905 Mr. Bunker was in Bolivia, South America, as Chief of Party on preliminary surveys, for the Bolivian Government, in the Andes Mountains. In 1905, he returned to the United States and for three years was employed in the South as Chief of Party and Resident Engineer on railroad work for the Louisville and Nashville, and Chesapeake and Ohio Railroad Companies. In the fall of 1908, he again went to South America, this time to Brazil, where as Division Engineer he assisted in the building of 250 miles of railroad in the Amazon Valley. The road commenced at the head of navigation in the Madeira River, 1 600 miles from the sea, and ended in the wilderness on the banks of the Mamoré River, the boundary between Brazil and Bolivia. This work was completed in 1912, at which time Mr. Bunker returned to the United States, going to Toledo, Ohio, where, for several months, he was engaged in street railway surveys.

In the fall of 1913 he accepted a position with the Maine State Highway Commission as Assistant Engineer. He was engaged in this work when the United States entered the World War. He immediately volunteered for service and was sent to the first Plattsburgh Camp in June, 1917. He was not without "peace-time" military experience as he had been Captain of the Color Company at the University of Maine and, later, a member of the National Guard of the State of Maine. Mr. Bunker was soon given a commission as Captain and sent to Fort Sheridan, Illinois, where he was a Drill Master.

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* Memoir prepared by Oliver L. Hall, Esq., Bangor, Me.

He went to France in November, 1917, as Captain of the 503d Engineer Regiment. His first service was in forestry work near the Swiss border, his company being engaged in manufacturing lumber for the trenches. He was then, during a period of several months, in charge of grading and steam shovel work in the Montoir Railroad Yards near St. Nazaire, the principal port of debarkation.

During his service in France Captain Bunker was also in charge of pouring the concrete for several of the immense warehouses constructed at St. Nazaire. Following the signing of the Armistice he was placed in charge of rebuilding the roads in this district, which were severely worn by army trucks.

Captain Bunker spent twenty months in war service in France, returning to the United States in July, 1919, in command of a detachment of the 72d Engineer Regiment. He proved himself an excellent officer, extremely efficient, and was held in the highest regard by his men. That his services were appreciated by his superiors is attested by the fact that he was recommended for promotion to the rank of Major, but unfortunately promotions were stopped by the Armistice. He later, however, received a commission as Major in the Reserve Officers' Corps.

Following his return from France Captain Bunker resumed his position with the Maine State Highway Department in charge of the construction of several concrete and gravel bituminous roads in various sections of the State. In January, 1923, by unanimous vote of the Board of Public Works, he was elected City Engineer of Bangor, Me., in which capacity he had charge of the street and highway, park, and sewer departments, with power to appoint his own subordinates. In his six years in this position, Captain Bunker was given credit for valued services. During this period the City constructed 10 miles of cement concrete roadway, 3 miles of cement concrete sidewalks, and 4½ miles of new sewers. In these years, 190 000 cu. yd. of gravel were used by the Municipal Government. This gravel was obtained on very advantageous terms through the purchase of a large gravel bank.

During his service as City Engineer, the Department was motorized from two pieces of apparatus in 1922 to twenty-five pieces of equipment in 1928, and, to-day, it is rated as one of the most competently equipped municipal departments in New England.

Captain Bunker always will be remembered in Maine as a pioneer in keeping adjacent highways free from snow during the winter. The movement spread by the co-operation of neighboring towns with the City of Bangor until the highways in that section were available for motor use all the year around. The effort was so successful that it was copied by other Maine communities until now there are few main highways in the State which are not cleared following heavy snows.

He was ever ready to extend aid and assistance to other City Departments as well as to local engineers and contractors who found the knowledge attained by him of vast help. Always courteous and kindly, he was held in high regard by the employees of the Engineer Department and by the citizens generally.

Captain Bunker died after an illness of several months in Bangor. The funeral services were held on December 17, 1928, at All Souls Congregational Church in that city. The interment was at Bar Harbor. He is survived by his widow, Euna (Sanford) Bunker, a native of Bath, Me., two daughters, Jeannette Mary, of Boston, Mass., and Ruth Elizabeth, and one son, Paul, in the employ of the Maine State Highway Department. He is also survived by his mother, Mrs. Jeannette Bunker, of Bar Harbor, and two sisters, Mrs. Maurice C. Rumsey, of New York City, and Mrs. Oliver L. Hall, of Hampden, Me.

Fitting testimonial to the worth of Captain Bunker as a public official was given by the following public announcement of his death made by the Hon. John Wilson, Mayor of Bangor:

> "Stephen S. Bunker, City Engineer of Bangor for seven years, after several months of failing health, died in Bangor, on Saturday, December 15, 1928. Mr. Bunker had served the city with faithfulness, honesty, and ability, and has left as his lasting memorial, a system of concrete roads on the main arteries of the city, of which the city is justly proud, and which establishes clearly his reputation as one of the great road engineers of New England. He was a loyal soldier in the Great War, a good citizen, and a man of high character with an exceptionally lovable disposition. The City of Bangor has met a great loss and will officially and sadly pay proper respect to his memory on the day of his funeral."

Captain Bunker was elected an Associate Member of the American Society of Civil Engineers on October 4, 1910, and a Member on June 2, 1920.

JAMES BURDEN, M. Am. Soc. C. E.*

Died March 1, 1928.

James Burden was born at Glasgow, Scotland on January 12, 1864, the son of John and Janet (Duncanson) Burden. When he was very young his parents came to the United States. He received his preliminary education in the public schools of Troy, N. Y., and at the Sterling, Scotland, High School; he was graduated from the Rensselaer Polytechnic Institute at Troy, in 1892, with the degree of Civil Engineer.

After his graduation from college, Mr. Burden made a specialty of construction work on dams, canals, and water-works. He was engaged for a time as Draftsman for the Metropolitan (Boston) Water-Works in connection with the Wachusett Aqueduct and Reservoir. In 1897 and 1898, he had charge of the New York State improvements on the Minetto Dam and on the canal at Manlius Center, N. Y. In 1899 he entered the employ of the Sharon Steel Company of Sharon, Pa., for which he designed and installed the sewers, water supply, and railroad yards at its new plant.

From 1900 to 1904, Mr. Burden served as Principal Assistant Engineer on the construction of the Troy Water-Works, in charge of the surveys and construction of the pipe lines and dams. From 1904 to 1908, he was Assistant Engineer in the main office of the New York State Barge Canal, at Albany, N. Y., where he made studies of back-water above proposed dams across the Mohawk River as well as of the design of locks and movable dams. In 1908 Mr. Burden was appointed Resident Engineer on various sections of the New York State Barge Canal, at Mechanicsville, Oswego, and Utica, N. Y., which position he retained until 1918. From 1919 to 1921, he was employed with the Emergency Fleet Corporation at Washington, D. C., and Philadelphia, Pa., in connection with dry docks and marine railways. The following year he was appointed City Engineer of Oswego, which position he held until his death.

Mr. Burden was a member of Grace Presbyterian Church of Oswego, and took an active part in church work, serving as President of the Board of Trustees. He was also a member of the Permanent International Association of Congresses of Navigation, the Fortnightly Club, and the Oswego Country Club. He was sincere in all his undertakings, giving his best to everything with which he was connected.

On December 5, 1912, Mr. Burden was married to Edith Sloan, the daughter of the late James and Mary Sloan, of Oswego, who survived him. Mrs. Burden died on October 23, 1928.

Mr. Burden was elected an Associate Member of the American Society of Civil Engineers on February 5, 1902, and a Member on January 5, 1904.

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* Memoir prepared by Morton Burden, Assoc. M. Am. Soc. C. E.

LUTHER HAROLD BURT, M. Am. Soc. C. E.*

Died January 29, 1929.

Luther Harold Burt was born on February 29, 1876, in Hartford, Conn., the eldest of seven children. His father, Luther W. Burt, was for many years in private practice as a Surveyor in Hartford, and for some time served as City Surveyor, so that the boy grew up in an engineering atmosphere. His mother, Mary A. Greene, of Middletown, Conn., and all his ancestors were of pure New England stock.

After receiving the usual preliminary education, principally in the Hartford Public Schools, he was graduated in 1900 from Trinity College, Hartford, with the degree of Bachelor of Science.

Following vacation work which began in 1895 with his father's field parties on various surveys and on the construction of the Springfield Branch of the Central New England Railway, Mr. Burt, in August, 1900, entered the employ of the Central Railway Company of New Jersey. He was, successively, Draftsman and in charge of a field party on maintenance of way for this Company, the Lehigh Valley, the Greenwich and Johnsonville, and the Pittsburgh and Lake Erie Railway Companies. For the latter, he was Draftsman and Designer in the office of the Chief Engineer on various structures connected with double- and four-tracking.

From March, 1903, to October, 1908, Mr. Burt was Draftsman and in charge of design under the Division Engineer of maintenance-of-way work for the Pennsylvania Railroad Company, at Philadelphia, Pa. The masonry design for the Delaware River Bridge above Easton, Pa., and the design of the West Morrisville Yard for the Pennsylvania Lines were among the larger items of his work during this period. Through the winter of 1905-06, he was Draftsman on the Electric Zone Improvements for the New York Central and Hudson River Railroad Company, and from March, 1906, until April, 1911, he was an Assistant Engineer for the Bronx Valley Sewer Commission, at White Plains, N. Y., in charge of important parts of the design and construction of large trunk sewers and other structures connected with the improvement of the Bronx Valley.

In April, 1911, Mr. Burt returned to Hartford and associated himself with his father, under the firm name of L. W. Burt and Son. After his father's death in 1921, he continued alone in the practice of surveying and general engineering.

He designed and supervised the erection of the Brazos Dam at Middletown, as well as the Baldwin-Ives Dam, and was in charge of many of the real estate sub-divisions in and about Hartford. He had systematized the records of his father's early work, and not only made great use of them in his work of re-establishing old land lines, but was generous in allowing others to use them.

Mr. Burt was a regular attendant at the meetings of the local engineering societies and was well liked by his associates and by all with whom he came in contact. His death removes an engineer who was a credit to the profession and a worthy citizen.

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* Memoir prepared by Henry R. Buck and Robert J. Ross, Members, Am. Soc. C. E.

He was married on April 7, 1903 to Claudia La Seur, of Schuylerville, N. Y., who survives him. Their only son, Richard White Burt, was associated with his father for some time and now carries on the work of the office under the old firm name.

Mr. Burt was elected an Associate Member of the American Society of Civil Engineers on November 1, 1910, and a Member on April 2, 1913.

CHARLES LINCOLN CARPENTER, M. Am. Soc. C. E.*

DIED SEPTEMBER 28, 1929.

Charles Lincoln Carpenter, the son of the Rev. Charles Carrol and Feronia (Rice) Carpenter, was born at Amherst, Mass., on June 17, 1867. On both sides of the family there was a background of sturdy New England stock, identified with the civil, religious, and martial life of the Colonies from their beginning. The first of the name in this country was William Carpenter, an English Puritan, who settled in Weymouth, Mass., in 1638. On the maternal side he was descended from Deacon Edmund Rice, an early settler (1638) of Worcester, Mass., and on the same side he was allied to the Knowlton and Pomeroy families.

Mr. Carpenter was graduated from Dartmouth College in 1887 and from the Thayer School of Civil Engineering in 1889. The funds for this college education were largely earned by himself in doing odd jobs around the campus and in working during the summer vacations.

After his graduation, Mr. Carpenter went to Nicaragua where he was engaged for two years for the Canal Construction Company as Leveler and Transitman on preliminary and railroad location surveys. He was in charge of 30 miles of telegraph line construction; detail surveys for locks and embankment of canal; and also of surveys on construction and improvements of Greytown Harbor.

In July, 1891, he returned to Massachusetts where he was employed with the Boston Board of Survey and the Street Commission of Boston for seven years, one year as Transitman and six years as Engineer in charge of surveys, calculations, and the laying out of a new street system in Boston.

In May, 1898, Mr. Carpenter went to Alaska on a prospecting and mining trip. On his return in November, 1900, he spent several months on preliminary and location surveys for the Boston and Worcester Electric Railroad Company, and then accepted a position in Cuba on railroad location and construction with the Cuba Central Railroad Company as Transitman and Assistant Division Engineer. At the expiration of a year in Cuba, he returned to the United States and was engaged for about three months as Assistant to the City Engineer of Gloucester, Mass. In September, 1902, however, he entered the employ of the Federal Government as U. S. Junior Engineer on river and harbor work at Boston.

When work on the Panama Canal was begun, Mr. Carpenter took one of the first parties to Panama in June, 1904, and the necessary surveys of the Great Gatun Lake Basin were started. Possible dam sites at Gamboa and Althajucta on the Chagres River, were investigated and reported on; and surveys for the relocation of the Panama Railroad at Bas Obispo, the location of the center line of the Canal on the Atlantic end, with the necessary borings, and the location of the Canal Zone boundary line, were all done under his supervision during the first two years.

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* Memoir prepared by E. F. Sinz, Assoc. M. Am. Soc. C. E.

In July, 1906, Mr. Carpenter was made Resident Engineer of the Gatun Dam and in July, 1907, Resident Engineer of the Gatun Locks, which position he held until July, 1908. All clearing of the site of this huge earth dam and these locks, surveys, borings for foundation, and preliminary excavations were made while he was in charge, and his work as an organizer was outstanding. The Porto Bello Quarry, where all the rock for the immense concrete structures on the "Atlantic end" was quarried and crushed, was put in operation under his direction.

In September, 1908, Mr. Carpenter again went to Cuba, this time for J. G. White and Company, in charge of the reconstruction of the Cuban Eastern Railroad. During the year of this contract, the road-bed was rebuilt so that it would stand up under traffic during the tropical rains and floods. He continued with this railroad, which was re-organized and named the Guantanamo and Western Railroad, as General Manager and built up the property into a prosperous organization.

In December, 1911, Mr. Carpenter went to Porto Rico as Superintendent of the Ponce and Guayama Railroad. In June, 1912, he was appointed to fill the vacancy of General Manager of the Central Aguirre Sugar Company and of the Ponce and Guayama Railroad Company. Later, the duties of Vice-President of these two companies were added as well as those of Vice-President and General Manager of The Central Machete Company and of the Santa Isabel Sugar Company. In 1919, he became one of the Managing Partners of Luce and Company, a land-holding organization in Porto Rico. He held all these positions until his death.

The professional career of Mr. Carpenter was a varied one and took him to many parts of the North American Continent—from Alaska to Panama. He was an outstanding administrator, as the progress of the works that he directed will testify. He demanded results, but he never asked his men to go where he would not go himself. During his work in the tropical jungles, he never spared himself, and this willingness to share all hardships won for him the loyalty and friendship of his associates, many of whom were always glad to work under him, wherever he happened to be. He was a great lover of the outdoors and his favorite form of vacation was a tramp through the New England mountains or a canoe trip along the rivers of Canada.

Death came to Mr. Carpenter very suddenly, while he was on one of these canoe trips in one of the wildest parts of the Upper Gatineau River region of Canada. A few days of hard paddling with several difficult portages were too much for his strength, and he died of heart failure on September 28, 1929. He will long be remembered by his associates as an able, beloved executive and by the poor as a generous giver when help was needed.

Mr. Carpenter was a Mason, and a member of various geographical and historical societies and of the Appalachian Mountain Club. He had contributed to the Byrd Antarctic Expedition and to many associations for the preservation of forests and of wild life. He cheerfully gave to the poor and distressed, particularly to children. Those who knew him well loved him for his whole-hearted friendship and kindness.

Mr. Carpenter was married on December 15, 1892, to Charlotte Florence Sullivan, of Boston, Mass., who, with three children, James Sullivan, Thomas Rice, and Charles Carrol, survives him.

Mr. Carpenter was elected a Member of the American Society of Civil Engineers on December 6, 1905.

JAMES FRANCIS CASE, M. Am. Soc. C. E.*

Died December 14, 1929.

James Francis Case was born at Columbus, Wis., on September 22, 1868, the son of the Rev. George Wilkinson Case and Nina (Nash) Case.

He was graduated from the University of Wisconsin in 1890. There followed a few years of railroad work in the Middle West; then came the Spanish-American War when he left Oregon as the Captain of a Company of Militia—was made a Major for gallantry and distinguished service in the Philippines; served as City Engineer of Manila; and, in 1908, was made Director of Public Works. During this period he did important work in the construction of harbors, roads, water-works, and sewers. He became proficient in the Spanish language and, later, a master of French, both of which were to be very helpful to him.

Major Case then became the Havana Representative of the Cuban Engineering and Contracting Company and, as its Vice-President, carried out the largest sanitary project the world had yet seen, building several hundred miles of sewers and drains and a very novel turbo-electric pumping station for that city.

In 1916 he joined the staff of the American International Corporation. He reported upon engineering projects of magnitude in Spain, Greece, Mexico, and South America, and also served as contact between his Company and the United States Government in the vast construction enterprises carried out by that Company during the World War.

For Ulen and Company Major Case negotiated many contracts in foreign countries, notably that for the construction of the water-works in Athens, Greece.

In 1928 he was appointed Paris Representative of Stone and Webster, Incorporated, which position he held at the time of his death. Taken ill with pneumonia while on a business trip in this country, he died in New York, N. Y., of heart failure, on December 14, 1929. He is survived by his widow, Helen (Smith) Case.

Born in a small town in the Middle West, this son of religious parents, with but little help, save his own indomitable will power, secured for himself an education, and, later, made for himself fame as an engineer, a reputation for kindness to his fellowmen, and an acquaintanceship which circled the globe. He was loved by bootblacks and bank presidents and was the friend of two Presidents of the United States, the Presidents of many Central American and South American countries, and of several of the crowned heads of Europe.

Major Case was chosen by the League of Nations to report on the transportation system of Poland. In 1918, he headed a delegation of American engineers that attended a Reconstruction Congress in France, at the invitation of the French Society of Civil Engineers.

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* Memoir prepared by Paul G. Brown, M. Am. Soc. C. E.

In 1929 he was awarded the James Laurie Prize by the Society for his paper "The Ancient Roman Aqueduct at Athens," which has been published* and is an outstanding contribution to engineering literature.

Major Case was a member of many clubs and societies, among which were: Western Society of Engineers, The Engineers' Club of New York, American Club of Havana, Havana Country Club, Havana Yacht Club, Vice-President, Società Italo Americana di Studi e Lavori Publici, Membre d'Honneur, Société des Ingénieurs Civils de France, Bankers Club of America, Sleepy Hollow Country Club, Army and Navy Club of Washington, D. C., and the Four Square Club, New York.

His life was a busy and a useful one, yet he was fond of sports and was helpful to his clubs. He served as a Governor of The Engineers' Club of New York for several years.

Major Case was elected a Member of the American Society of Civil Engineers on January 6, 1904.

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* *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 281.

EDWARD BERTIE CODWISE, M. Am. Soc. C. E.*

Died February 11, 1927.

Edward Bertie Codwise, the son of Elisha Rogers and Harriette M. S. (Salter) Codwise, was born at Elizabeth, N. J., on May 9, 1849. His ancestors came to the United States from Holland in 1624, and settled in Massachusetts. In 1664 the family moved to Elizabeth. His great-grandfather, Christopher Codwise, was a field officer in the Continental Army and a member of the Society of the Cincinnati.

Mr. Codwise received his education at the Baldwin Academy, in New York, N. Y., from which he was graduated. Later, he attended the Brooklyn Collegiate and Polytechnic Institute and was graduated with the Class of 1865. Subsequently, he was for two years a student at the L'Institution Davaux, at Amiens, France.

On his return to the United States in 1868, Mr. Codwise accepted a position as Assistant Engineer with the Erie Railroad Company on the construction of the Newburgh and New York Railroad, the Fort Lee Railroad, and the Weehawken, N. J., Cattle Yards. From 1869 to 1870 he served as Assistant Engineer with the New Jersey Southern Railroad Company, and from 1870 to 1872 he was engaged as Principal Assistant Engineer for the Wallkill Valley Railroad Company on construction work from Gardner to Kingston, N. Y.

Mr. Codwise spent a brief period from March, 1872, to April, 1873, in Peru, South America, where he was Assistant Engineer on the Callao, Lima, and Oroya Railroad. He was engaged for part of this time on the design and construction of what proved to be, for many years, the longest and highest railroad bridge in the world.

After his return to this country, he held for a short period the position of Principal Assistant Engineer for the New Jersey Southern Railroad Company. In 1874, he opened a private office in Rosendale, N. Y., and also served as Engineer for the Rosendale Cement Company. In 1876 he gave up this occupation to accept the position of Assistant Engineer in charge of inventory surveys, etc., for the Erie Railroad Company.

From 1879 until his retirement in 1923, Mr. Codwise held the following positions: Assistant Engineer in charge of the construction of the New York City Elevated Railroad on Ninth Avenue between 30th and 59th Streets; Division Engineer for the Albany and Jersey City Railroad (now the West Shore Railroad) Company on location surveys; Assistant Engineer in charge of the construction of extensions of the Wallkill Valley Railroad; Resident Engineer of the West Shore Company in charge of construction from Newburgh to Kingston, N. Y.; Chief Engineer of the Ulster and Delaware Railroad Company; and City Engineer of Kingston, where he was also engaged in private practice. During this time, Mr. Codwise designed the present water supply system of Kingston. His choice for the location of the reservoir was between two sites, one of which contemplated that a dam be placed across Esopus Creek only a few hundred feet down stream from the site of the present Ashokan Dam of the New York Water Supply. The reservoir, however, was constructed at the other location on Sawkill Creek, which was much nearer Kingston and more economical.

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* Memoir prepared from information on file at the Headquarters of the Society.

Because of a complete collection of original maps of Central and Eastern New York, which date back to the land grants from European rulers, Mr. Codwise was called upon to compile the location of the lands and boundaries of the State Forest Preserve.

He was married on March 28, 1872, to Emma Snyder, of Rosendale, who died in 1924. After his wife's death, he resided with his daughter, Mrs. R. H. Edmondson, of Morgantown, W. Va., who, with his two sons, Henry R. Codwise, M. Am. Soc. C. E., and George Wallace Codwise, also a Civil Engineer, of Kingston, survives him.

Mr. Codwise was exceedingly fond of his family, preferring his home to clubs and social life. He was of a happy, likeable disposition, admired by all his employees and honored by his business associates. His reputation for honesty and integrity was well demonstrated by the confidence that was placed upon his testimony when he appeared as an expert witness in Court cases. He was a member of the Kingston Club and of Kingston Lodge No. 10, F. and A. M.

Mr. Codwise was elected a Member of the American Society of Civil Engineers on September 5, 1888.

EDWARD ZANE COLLINGS, M. Am. Soc. C. E.*

Died November 19, 1928.

Edward Zane Collings was born on a ranch near Beaver City, Nebr., on December 12, 1885. He was graduated from the High School of Beaver City in 1902 and entered the State University of Nebraska the same year, taking a course in Civil Engineering. Circumstances prevented his completing the course, however, and, in 1905, he moved to California.

Mr. Collings began his practical engineering experience with the City Engineer of Santa Monica, Calif., following a short period spent with the Los Angeles and Salt Lake Railroad Company. In 1906 he moved to Los Angeles, Calif., where he entered private practice with Mr. F. D. Lanterman on subdivision work. This engagement was not of long duration.

Subsequently, he entered the service of the City of Los Angeles and, with the exception of a short period of absence, he remained in this service until the beginning of the World War in 1917. During this association Mr. Collings advanced rapidly through the stages of Draftsman and Grade Computer in the office, and of Instrumentman, Chief of Party, and Assistant Engineer in the field. He had charge of the surveys for the annexation of the Towns of San Pedro and Wilmington to the City of Los Angeles, and was the first Harbor Chief Surveyor when the initial parties were put in the field by the Harbor Department after the annexation. The field surveys of the present triangulation system covering the Harbor District were made under his supervision. In fact, he was one of the first of the City's engineers to pioneer in the development of Los Angeles Harbor. Many of the present pier-head lines, wharfs, and municipal docks were established during his service as Harbor Surveyor. He was greatly interested in water-front work of all kinds and during his leisure he studied navigation and obtained a navigator's license.

As Civil Service Assistant Engineer, Mr. Collings established and operated a rock quarry on Catalina Island for the Los Angeles Harbor Department, the quarried rock being transported and used in jetty construction.

While in college Mr. Collings had had military training which stood him in good stead when the United States entered the World War. Being an officer in the Reserve Corps, he was one of the first to register at the Presidio at San Francisco, Calif., and to become a Captain in the 316th Engineers, 91st Division, stationed at Camp Lewis, Washington. It was in the Battle of the Argonne, and while in command of his Company, that he was severely wounded. Eventually, he lost one of his legs after spending three years of suffering in a hospital in an attempt to save it. Captain Collings was cited for bravery in action and recommended for a Majority. After the Battle of the Argonne, his wound resulted in disablement as far as the Army was concerned.

Army hospital experience cost Mr. Collings dearly, necessitating inactivity in his profession. Returning to Los Angeles, in 1921, he was reinstated with the City of Los Angeles as a Designing Engineer in the Street Planning Division. In this capacity he remained until the summer of 1923. Being recognized as proficient in this class of work, the Strong and Dickinson Syndicate of Los Angeles employed him as Chief Engineer on municipal subdivision and street improvements, and as such he served that Company under a Deputyship of the City Engineer's Office. The subdivision, planning, and improvement of the Morena Highlands and Silver Lake Terrace Tracts are monuments to his engineering ability. Engaged for a period of more than six years with the same firm, he became recognized as an authority on city planning and development.

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* Memoir prepared by Sherman A. Jubb, M. Am. Soc. C. E., and W. T. Collings, Jr., Assoc. M. Am. Soc. C. E.

For three years previous to his death, Mr. Collings served as President of the Northwest Chamber of Commerce in the City of Los Angeles. In this capacity, he was most active in the development of the northwest section of the city and in addition was a leader in civic affairs. Mr. Collings' death resulted directly from an operation which he underwent with the hope of being restored to better health.

Mr. Collings was married, in 1906, to Elsie Verna Dakan, formerly of Beaver City, Nebr., whom he had known from childhood. His widow, one son, and two daughters survive him.

The achievements of Mr. Collings, particularly for the time dating from his discharge from the Army, are noteworthy, considering his physical handicap. During those years and until the time of his death, although suffering almost constant pain, he was ever active. Civic, political, and military organizations demanded much of his time. He was active among World War Veterans, and was Commander of the Disabled Emergency Officers of the World War and a member of the Sunshine Post of the American Legion. He was also a member of the First Presbyterian Church of Glendale, Calif., and keenly interested in all its affairs. Genial in disposition, a loving father and friend, he will be missed by all who ever knew him or worked with him.

Mr. Collings was elected a Member of the American Society of Civil Engineers on March 7, 1921.

GEORGE BIRDSALL CORNELL, M. Am. Soc. C. E.*

DIED MARCH 14, 1929.

George Birdsall Cornell was born in New York, N. Y., on October 17, 1855, the son of Birdsall Cornell who founded the old Cornell Iron Works and served as its head for a number of years. His family came to America in 1638 from England and first settled on Long Island.

Mr. Cornell received his early education abroad, attending private schools in Switzerland and France, and preparing for college at Anthem's School in New York. He was graduated from Columbia College, New York, in the School of Mines, in 1877, with the degrees of Engineer of Mines and Civil Engineer.

Mr. Cornell began his professional career as Rodman with the Engineer Corps of the Manhattan Elevated Railway Company, and in 1879 was Assistant Engineer on the construction of the Brooklyn Elevated Railway. From 1880 to 1882 he served as Engineer on the location and construction of the Buffalo, Rochester, and Pittsburgh Railway in Pennsylvania. Subsequently, he was engaged for two years as Bridge Engineer with the late Walter Katté, M. Am. Soc. C. E., during the construction of the New York, West Shore, and Buffalo Railway.

In 1884 he became Chief Engineer for the Brooklyn Elevated Railway Company, during the time of the construction and equipment of its lines for operation. Mr. Cornell was then employed as Chief Engineer of Construction for the Union Elevated Railway Company, Brooklyn, N. Y., until 1889, when he went to Chicago, Ill., as Chief Engineer on the construction of the Chicago South Side Elevated Railway. He returned to New York in 1892, as Chief Engineer of the J. B. and J. M. Cornell Iron Works, engaged on what was, at that time, high building construction.

During the period extending from 1893 to 1895, Mr. Cornell was Chief Engineer for the East River Bridge Company and prepared the plans for a bridge on the site of the present Williamsburgh Bridge, obtaining the approval for this undertaking from the War Department. In 1895, he again took up the duties of Chief Engineer for the Brooklyn Elevated Railway Company and of Chief Engineer and General Manager for the New York and Brooklyn Bridge Railway Company, in which capacity he was in charge of the operation of bridge trains. He prepared and obtained the approval of plans for the elevated railway to cross the East River Bridge, and put these lines in operation. He also installed the first electric train service on the Brooklyn Elevated Railway. In 1898, Mr. Cornell went to Europe to investigate automobile designs for New York capitalists.

He was engaged on consulting work, from 1899 to 1903, reporting on water power, railway, and electrical developments in the United States and abroad. Among other projects was one for building an elevated railway along the route of Regents Canal in London, England. In 1904, he again was called abroad to report on electrical problems in France and Germany for international bankers.

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* Memoir prepared by H. J. Cornell, Esq., New York, N. Y.

Returning to the United States, Mr. Cornell, in 1905, made a reconnaissance for a railroad from Quebec, Que., Canada, to the French River, covering the entire route in winter on snowshoes. In 1906, he went to Cuba, to report on electrical and power properties for Cuban bankers and sugar interests and from 1906 to 1907 he was employed as Chief Engineer with the Memphis Street Railway Company, and as Resident Engineer for Ford, Bacon, and Davis, covering construction and reconstruction of railway, power plant, and overhead lines. In 1908 he became Chief Engineer for Meikleham and Dinsmore which firm he served until 1914 in charge of the design and construction of steam and hydro-electric plants, high-tension transmission lines, and electric railways in New York, New Jersey, and Pennsylvania.

From 1914 to 1918, Mr. Cornell was engaged in the supervision of power plants and distribution systems and was also associated with J. C. Brackenridge, M. Am. Soc. C. E., on the valuation of public utilities and steam railroad properties for the Cities of New York, Jersey City, N. J., Hoboken, N. J., Buffalo, N. Y., Little Falls, N. Y., and others. From 1918 to 1921 he served with Ford, Bacon, and Davis, first as Resident Engineer on a marine boiler plant at Richmond, Va., for the U. S. Shipping Board. He later acted as Engineer of Design of the factory and laboratory for the Pathé Company and as Valuation Engineer on properties of the Public Service Corporation of New Jersey and of other public utilities.

From 1921 to 1924, Mr. Cornell was again associated with Mr. Brackenridge on plans for the station of the Hudson and Manhattan Railway Company at West Side Avenue, Jersey City, and on testimony before the Public Utilities Board of New Jersey for Jersey City. He also designed the station at Ridgefield Park, N. J., for the New York, Susquehanna, and Western Railway Company, and offered testimony before the Public Utility Commission of New Jersey for the Town of Ridgefield Park. In behalf of adjacent property owners, he assisted in the valuation of the 42d Street Spur of the New York Elevated Railways, looking to its removal, and from 1924 to 1926 was engaged in the valuation of the Yonkers Electric Light and Power Company, and of the New York Edison Company for Murray and Company. The following year he was employed on the valuation of the property of the Philadelphia Electric Company for Gillette and Malcomson.

Mr. Cornell retired from active practice in 1928 after fifty-one years of varied and successful achievements in the Engineering Profession. He was a member of Psi Upsilon Fraternity and a Thirty-second Degree Scottish Rite Mason.

On January 31, 1882, he was married to Eleanor Jackson, of Ridgway, Pa. He is survived by a daughter, Mrs. James B. Tailer and five sons, George B., Jr., Harold Jackson, Robert Leslie, Thomas Whilehead, and Francis Shepard Cornell.

Mr. Cornell was elected a Junior of the American Society of Civil Engineers on August 6, 1879, and a Member on October 6, 1886.

ALFRED CRAVEN, M. Am. Soc. C. E.*

Died September 30, 1926.

Alfred Craven was born in Bound Brook, N. J., on September 16, 1846, the son of the late Rear-Admiral Thomas Tingey Craven, U. S. N., and Emily (Henderson) Craven.

His forebears served in the Navy in every generation since and including the American Revolution. He was a descendant of Thomas Tingey, a Captain of the Continental Navy. His father commanded the Sloop of War, *Brooklyn,* of Farragut's Fleet in the passage of Forts Jackson and St. Philip on the Lower Mississippi on April 24, 1862. An uncle, Tunis Augustus Macdonough Craven, was in command of the monitor, *Tecumseh,* of Farragut's Fleet at the Battle of Mobile Bay, on August 5, 1864. The *Tecumseh* struck a torpedo and sank in 30 sec. with most of her crew. Commander Craven was in the pilot-house with the pilot when the ship struck. The opening from the pilot-house would permit but one at a time to pass. While realizing that every second counted, Commander Craven turned to the pilot and said, "You first, Sir." The pilot was saved and the Commander was drowned.

Following the traditions of the family, Alfred Craven entered the United States Naval Academy at Annapolis, Md., during the Civil War and was graduated in 1867, remaining in the Service until 1871 when he retired with the rank of Master. In 1871, he joined the California Geological Survey which did such notable work for the State. He was a member of the famous Whitney Survey and was one of the party of three under Mr. C. F. Hoffman who were the first to climb to the summit of Tower Peak. Later, he was engaged on irrigation work in the Sacramento and San Joaquin Valleys and then went into private practice in Virginia City, Nev., where he was connected with the development of the famous Comstock Lode. On this work, he was associated with Adolph Sutro in the construction of the well-known Sutro Tunnel designed to drain some of the deep mines.

Mr. Craven returned East in 1884 and became an Assistant Engineer on the surveys for, and the construction of, the New Croton Aqueduct of the New York Water Supply, serving under the Aqueduct Commissioners. This was a work of great magnitude for those days. The aqueduct was more than 30 miles long, constructed almost entirely in rock tunnel, and lined with brick. It was designed for a capacity of about 300 000 000 gal. daily. When the work was placed under contract in the early part of 1885, Mr. Craven was made Division Engineer of the Fourth Division.

On this work he established an enviable record for ability, courage, and rugged honesty. He stood firmly against practices resulting in dishonest work. After some straightforward and unimpeachable testimony before an investigating committee, an incensed politician is said to have told Mr. Craven: "You have done the last piece of work you will ever do on this job"; whereat, Mr. Craven quietly replied, "I will be here when you are gone."

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* Memoir prepared by Robert Ridgway, Past-President, Am. Soc. C. E.

This proved to be correct, for the agitation led to a legislative investigation and the passage of an act which permitted Mayor Hewitt to re-organize the Aqueduct Commission and to appoint men of a caliber to appreciate honest service. Because of his fine record and his ability as an engineering executive, Mr. Craven was given increased responsibility and he assisted in completing the aqueduct in a manner that reflected credit on himself and others concerned.

For more than fifteen years Mr. Craven was a Division Engineer on the construction of this aqueduct and additional reservoirs of the Croton System. He was in charge of the construction of Reservoir "D" on the West Branch of the Croton, near Carmel, N. Y. (capacity about 9 000 000 000 gal.) and of the completion of Reservoir "M" on the Titicus River, near Purdys, N. Y. (capacity more than 7 000 000 000 gal.). These are two of the chain of storage reservoirs built before and since in the Croton Basin, and they are still in service. Between 1895 and 1900, he was in charge of the building of the Jerome Park Reservoir, a storage or equalizing reservoir of the Croton System, located in the Borough of The Bronx, New York City. During the progress of this work, material changes were made in the plans, of which he did not approve, and early in 1900 he resigned.

In May of the same year, when the first New York City Rapid Transit Subway work was begun, Mr. Craven was appointed Division Engineer on the Staff of the Rapid Transit Railroad Commission of New York City, and was given charge of the construction of that part of the four-track subway from 41st Street and Park Avenue to Broadway and 104th Street, known as the Second Division. He was appointed Deputy Chief Engineer of the Commission in 1904; Deputy Engineer of Subway Construction of the Public Service Commission (the successor of the Rapid Transit Railroad Commission) in 1907; Acting Chief Engineer of the same Commission in 1910; and Chief Engineer in 1911.

Under Mr. Craven's administration as Chief Engineer, the great so-called "Dual Subway System", costing for construction and equipment more than $500 000 000, was planned and, in a great part, built. At that time this was one of the greatest municipally constructed projects ever undertaken, comparable in magnitude with, and greater in cost than, the Panama Canal, and presenting great difficulties in planning and construction. That work may fittingly be regarded as a monument to Mr. Craven. He resigned as Chief Engineer in 1916, and was immediately made Consulting Engineer of the Commission holding that position under the Commission, and its successor the Transit Construction Commissioner, until he retired from service on December 31, 1920.

Mr. Craven was one of those men who, with no apparent effort on his own part, won the respect and affection of all who served under him or were otherwise associated with him. He never sought popularity. He had but to request that a thing be done and his Staff would work cheerfully day and night to accomplish the result. His fairness and straightforward way of thinking appealed to those who had business with him, and his decisions were generally accepted, whether they were for or against the claimants.

In commenting on his death, former Transit Commissioner Le Roy T. Harkness said:

"Mr. Craven was one of those really great men whose outstanding work was only thoroughly appreciated by the Engineering Profession and by those who had the rare pleasure of being associated with him. The people of New York use the magnificent subway system designed and built by Mr. Craven, but only a handful know of him. For this Mr. Craven's modest and retiring disposition was, of course, responsible. He was blunt and straightforward with a courage and integrity that matched his great ability. It has been said of him that he had no hinge in his moral backbone. Engineers generally realize that, while the Panama Canal is comparable with the New York Subway System in the matter of cost, there is no comparison in the matter of difficult and perplexing engineering problems. In the minds of those who knew and worked with Mr. Craven, the great Dual Subway System will always be regarded as a great monument to him. I had the rare privilege of serving with Mr. Craven for nearly ten years, and during that time came to have the same admiration and affection for him that was universal among all those who ever worked with or for him."

The Memorial Resolution passed by the Board of Transportation of the City of New York on October 5, 1926, reads, in part, as follows:

"This Board has learned with deep regret of the death of Alfred Craven on September 30th, 1926.

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"His memory will be a bright spot in the hearts of all who knew him. His sound judgment and absolute fairness, combined with his kindliness and modesty, won for him the respect and admiration of commissioners and contractors alike. His subordinates gave him the willing, loyal, and unstinted services that such a character inspires.

"His work is his monument."

Mr. Craven was survived by his widow, Nina Florence Browne Craven, to whom he was married in California in 1871, and who died in 1928. He is survived by three daughters, Mrs. Lucy Edgerton Bruce, Mrs. Emily Henderson Whittlesey, and Mrs. Nina C. Jones.

Mr. Craven was fond of fishing and golf. He was one of the first to take up the latter game in this country, and was a member of the St. Andrew's Golf Club. He was a member of the Society of Municipal Engineers of the City of New York, the Navy Athletic Association, and the Senior Golf Association. In 1915, he was appointed one of the two representatives of the Society on the Naval Consulting Board and served on it until his death. In 1908, he was awarded, under an Act of Congress, the Civil War Medal.

Mr. Craven was elected a Member of the American Society of Civil Engineers on December 5, 1888. He served as Director from 1903 to 1905 and as Vice-President in 1916 and 1917.

CARLTON ROLLIN DART, M. Am. Soc. C. E.*

Died June 23, 1929.

Carlton Rollin Dart, the son of Rollin C. and Sarah E. (Darling) Dart, was born at Lansing, Mich., on February 1, 1862. He was graduated from the Michigan Agricultural College in 1881 with the degree of Bachelor of Science. He began his professional career in 1881 as Assistant to the City Engineer of Lansing, but entered the University of Michigan in September, 1882, for the autumn and winter semesters.

In May, 1883, Mr. Dart again took up engineering work as a Surveyor and Draftsman on construction for the Marquette and Western Railway Company, at Marquette, Mich., making plans for a 50-pocket ore dock which was built at that place. From June, 1885, to September, 1886, he was Draftsman for the Grand Rapids and Indiana Railroad Company, at Grand Rapids, Mich.; and from September, 1886, to May, 1887, he was Assistant Engineer on construction for the Kansas City and Omaha Railroad Company, in Nebraska. He then entered the service of the St. Joseph and Grand Island Railroad Company, at St. Joseph, Mo., and was in charge of the construction of the railroad shops and terminal yards built for the St. Joseph Terminal Railway Company. In March, 1889, he entered the employ of the Union Pacific Railroad Company and was engaged on the work of building shops and terminal yards at Cheyenne, Wyo., and from May, 1890, to February, 1891, he designed and built railroad terminals for the Oregon Short Line and Utah Northern Railroad Companies, at Salt Lake City, Utah.

In February, 1891, Mr. Dart located in Chicago, Ill., and served until June of that year on building work in that city, after which he joined the Engineering Corps on construction work for the World's Columbian Exposition and was thus engaged until June, 1893. In July, 1893, he began the practice of general engineering with an office in Chicago and made complete plans for a large power station for the North Chicago Street Railway Company. In March and April, 1896, he was engaged on the structural work on the bear-trap dam at Lockport, Ill., collaborating with the late Alfred Noble, Past-President, Am. Soc. C. E., in this work. From May to December, 1896, he represented the late George S. Morison, Past-President, Am. Soc. C. E., in making surveys for a water-power project in East Tennessee, and from February, 1897, to January, 1898, he served as Draftsman in the office of Mr. Morison in Chicago. He then entered the service of the Lassig Bridge and Iron Works of Chicago, as Structural Draftsman and Checker, which position he retained until July, 1900, from which time until February, 1901, he was engaged in making drawings under the direction of Mr. Noble for compensating works in St. Mary's River at Sault Ste. Marie, Mich.

In February, 1901, Mr. Dart entered the service of the Sanitary District of Chicago as Assistant Engineer in charge of the Bridge Division. In December, 1902, he was promoted to be Bridge Engineer and served in that capacity until

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* Memoir prepared by Joint Committee of the Illinois Section and the Western Society of Engineers, consisting of Langdon Pearse, Albert F. Reichmann, and George M. Wisner, Members, Am. Soc. C. E.

March, 1921, when he was appointed Consulting Bridge Engineer, which position he held until his death.

Under Mr. Dart's direction were built many of the larger bridges over the Chicago River and practically all the bridges over the Drainage Canal, among which were the State Street Bridge, the Dearborn Street Bridge, the Jackson Boulevard Bridge, the Twenty-Second Street Bridge, the Western Avenue Bridge, the Eight-Track Bridge, etc. He also designed the butterfly dam at Lockport, the controlling works for the Calumet-Sag Channel, as well as all bridges over the North Shore Channel and the Calumet-Sag Channel. He was also engaged as a Consultant on the structural work carried on at the various pumping stations and other buildings of the Sanitary District, and took a keen interest in the development of the proposed controlling works suggested for the Chicago River.

Personally, Mr. Dart was a very kindly man, extremely modest as to his own accomplishments, and very considerate of his associates, as well as helpful in emergencies. During his later years, he had taken an interest in gardening.

He was a member of the Western Society of Engineers, of which he served for many years as Treasurer, the American Association of Engineers, the American Association for the Advancement of Science, the Structural Engineers' Association of Illinois, the American Railway Engineers' Association, and the Geographic Society of Chicago. He was also a member of the Engineers' Club.

In 1908, he was married to Ella Weinland, who died several years before him.

Mr. Dart was elected a Member of the American Society of Civil Engineers on May 6, 1903.

FREDERICK AUSBERT DELAVAU, M. Am. Soc. C. E.*

Died May 28, 1929.

Frederick Ausbert Delavau was born on August 25, 1871, in Rochester, N. Y., the son of Joseph Stansbury Delavau and Margaret (Clements) Delavau.

Mr. Delavau's entire life was spent in Rochester, where he likewise attended school. In 1889, he was employed by Gray and Storey, Civil Engineers, and subsequently, with William R. Storey, Civil Engineer, and with Mr. William G. Gray, on surveys and in private practice.

In 1898 he was employed by the City Engineer of Rochester as Draftsman and Transitman, from which positions he rose steadily through the various grades of Assistant Engineer and Office Engineer. In 1926, Mr. Delavau became Deputy City Engineer and, in 1928, Supervising Engineer in Charge of Local Improvements. He retired on a disability pension on April 16, 1929, after thirty-one years of service. After a year's illness he died at the Strong Memorial Hospital on May 28, 1929.

Mr. Delavau was a conscientious worker and led an exemplary life. In 1903, he was married to Sallie E. Williamson, of Skaneateles, N. Y., and is survived by his widow and one son, Robert W. Delavau, both residing in Rochester.

He was a member of the Masonic Order and the Shrine, the Rochester Engineering Society, and the American Society for Municipal Improvements.

Mr. Delavau was elected a Member of the American Society of Civil Engineers on January 17, 1927.

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* Memoir prepared by C. A. Poole and John F. Skinner, Members, Am. Soc. C. E.

EUGENE FRANK DELERY, M. Am. Soc. C. E.*

Died November 24, 1929.

Eugene Frank Deléry, the son of Frank F. and Cora (Hall) Deléry was born in New Orleans, La., on November 26, 1877. Educated in New Orleans at Farde's Private School, and, later, at Tulane High School and Tulane University, Mr. Deléry was graduated from the latter in Civil Engineering in 1899, and in Mechanical Engineering in 1900.

From July to September, 1899, Mr. Deléry served as Rodman and Computer with the Mississippi River Commission on levee surveys and erection; during April and May, 1901, on design and irrigation plant and boiler settings, for A. M. Lockett and Company, New Orleans; and from May to December, 1901, as Draftsman for the Johnson Iron Works, New Orleans, on steel hull construction of the Southern Pacific tugs, *El Vivo* and *El Listo,* boiler piping, and general machinery design.

From December, 1901, to September, 1903, Mr. Deléry was engaged in private design work, chiefly for the following firms: The Shakespeare Iron Works, the John N. Murphy Iron Works, and with Frank Paul Barber, New Orleans. From September, 1903, to June, 1904, he served as Manager of the St. Louis Rice Mill. In July, 1904, he entered into private design practice and general machinery design. From January, 1905, to August, 1908, he was Surveyor and Construction Engineer at the U. S. Naval Station, New Orleans.

From 1908 to the date of his death, Mr. Deléry served as Assistant Engineer of the Sewerage and Water Board of New Orleans. His most important piece of work done in this connection was in the capacity of Designing Engineer on the Purification Plant Extension.

His contemporaneous private practice covered irrigation plant design, settling tanks for settling out by-products, and miscellaneous machinery design. He had acted as Consulting Engineer for the architectural firm of Emile Weil, Incorporated, on the construction of the new Saenger Theatre and the Tulane University Stadium. At the time of his death he was associated with the J. F. Coleman Engineering Company on the design of filter plants. He was also associated as Consulting Engineer for A: Merrick Blamphin, Assoc. M. Am. Soc. C. E., on drainage work in St. Bernard Parish, Louisiana, and with Henry A. Mentz and Company, Consulting Engineers, of Hammond, La., on the design of plants for water filtration.

Mr. Deléry's mechanical and engineering ability resulted in the invention of the rotary flow pump and impact strainer system. He also had patents on the following appliances: Axial flow pump, the hydraulic thrust bearing, and a machine for forming the depressed orifices in the impact strainer tubes, with other patents on hydraulic appliances pending in the Patent Office.

The impact system of filter underdrains, of which he was the inventor, has proved very successful and is in use in the 72 000 000-gal. addition to the New Orleans Filtration Plant, and has been installed in the 24 000 000-gal. addition to the Tulsa, Okla., Filtration Plant. They have also been installed at

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* Memoir prepared by Henry A. Mentz, President, Henry A. Mentz, Inc., Hammond, La.

Franklin, Donaldsonville, Monroe, and Lafayette, La., and Wemoka, Okla., and at the time of Mr. Deléry's death, were being considered for several large cities.

Mr. Deléry was a member of the American Society of Mechanical Engineers, and the Louisiana Engineering Society. He was also a devout member of the Roman Catholic Church, and of the Society of the Blessed Virgin of the Church of the Immaculate Conception, New Orleans.

Stricken in the prime of life, he died on November 24, 1929, of a cerebral hemorrhage, due to high blood pressure and heart trouble, in the chapel of the Ursuline Convent, where he had stopped for a moment in prayer.

On July 9, 1918, Mr. Deléry was married to Edna Marie Fuselier. He is survived by his widow, and two small children, R. de la Claire and E. Marie Yvonne, also by two brothers, Clarence H. and Frank B. Deléry, and five sisters, the Misses Caro, Edna, and Cora Deléry, and Mrs. C. A. Shaw and Mrs. Freret.

A man of charming personality and lovable character, Mr. Deléry was admired by all who came in contact with him, for his great ability as an engineer, and for his many endearing qualities as a man.

His professional work was marked by a high degree of integrity. His death is deeply deplored by all his associates, by whom he was held in the highest esteem and who hold his memory in affectionate regard.

Mr. Deléry was elected an Associate Member of the American Society of Civil Engineers on May 7, 1913, and a Member on March 16, 1925.

FREDERIC LESLIE DUDLEY, M. Am. Soc. C. E.*

Died March 26, 1929.

Frederic Leslie Dudley was born in Templeton, Mass., on August 16, 1858. He attended the public schools of Templeton and studied Civil Engineering at the Worcester Polytechnic Institute, at Worcester, Mass., from which he was graduated in 1879.

After leaving the Institute, Mr. Dudley spent more than a year as Draftsman with Edward P. Adams, Landscape Architect and Civil Engineer, of Boston, Mass.

This was the period of railroad expansion in the West and in February, 1881, Mr. Dudley went to Kansas, where he was engaged in railroad construction for a number of years, first with the Engineering Corps of the Atchison, Topeka, and Santa Fé Railroad Company and the St. Louis, Iron Mountain, and Southern Railway Company; and, later, as Division Engineer of the Burlington and Missouri River Railroad Company on construction work in Nebraska and Colorado.

In 1888, Mr. Dudley returned to the East and entered the employ of the Edge Moor (Del.) Bridge Company. In this new line of work he started as Draftsman, then became Checker, and, in two years, was in charge of design as a Squad Leader. He supervised the detailing of all classes of structures; bridges, however, became his chosen specialty. At this time he began his work on swing bridges which later developed into all classes of movable bridges, in which field he became well known. He contributed much valuable information of a permanent nature to this branch of the Engineering Profession.

In 1901, the Edge Moor Bridge Company became a part of the American Bridge Company and in January, 1904, Mr. Dudley was transferred to the new Ambridge, Pa., Plant of this Company, in charge of the detailing of movable bridges. In 1905, he became Engineer of the Drawing Room for Railroad Bridges, including the design of machinery for movable bridges. For a number of years previous to his death his work also included the development of self-contained, power-driven railroad turn-tables and machinery for special movable structures, such as transfer-tables, ore and coal-handling bridges, etc. Until his retirement on January 1, 1929, just three months before his death, he continued in charge of this special work at the Ambridge Plant and as Consultant in the design of machinery for many movable structures.

During his twenty-five years at the Ambridge Plant, Mr. Dudley supervised the detailing of more than one hundred movable bridges of all types, including many of the larger swing and lift bridges, built by the American Bridge Company, in the United States. While in charge of the Drawing-Room for Railroad Bridges he supervised the detailing of many large structures, among which were the Mobile River Bridge, for the Louisville and Nashville Railroad Company (330-ft. draw span); the Hackensack River Bridge, for the Delaware, Lackawanna, and Western Railroad Company (a three-track, 198-ft. vertical lift bridge at Jersey City, N. J.); the Tombigbee River Bridge, for the

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* Memoir prepared by R. G. Manning, M. Am. Soc. C. E.

Southern Railroad Company (184-ft. vertical lift span); the Rigolets Bridge, for the Louisville and Nashville Railroad Company (414-ft. draw span); the coal-handling bridge for the Carnegie Steel Company, at Clairton, Pa. (a double trolley, 400 ft. between shear leg and tower, 630 ft. over all, including the mechanical and electrical equipment).

The fixed long-span bridges on which Mr. Dudley was engaged, included a bridge for the Kentucky and Indiana Terminal Railroad Company, at Louisville, Ky. (two 615-ft. spans and one 400-ft. draw-span erected as cantilevers from adjoining spans, respectively, 275 and 373 ft.); the Ohio Connecting Railway Bridge, for the Pennsylvania Railroad Company, over the Ohio River, at Pittsburgh, Pa. (spans, 416 and 525 ft., respectively, each half of the latter erected as a cantilever using one-half of the former erected temporarily at each end as anchor arm); the suspension bridge over the Ohio River between Parkersburg, W. Va., and Belpre, Ohio (three spans, 775, 375, and 275 ft., and approaches); the 977½-ft. arch span for Hell Gate Bridge (six tracks) over the East River from Long Island to Ward's Island, New York, for the New York Connecting Railroad Company; and the single-track cantilever bridge over the Ohio River at Steubenville, Ohio (660-ft. channel span and 230-ft. anchor arms). Many of these bridges involved very special and difficult erection features, the details of which had to be developed in connection with the steel work.

While Mr. Dudley never entered the field as an author of technical books, he contributed valuable information to technical students, draftsmen, and engineers, in the form of a set of blue-print notes (54 pages) entitled, "Notes on Engineering Bridge Details." This pamphlet was widely distributed to college students and instructors in structural courses and to employees in the structural drawing-rooms all over the United States and Canada.

In the course of his practice, Mr. Dudley acquired a large amount of valuable information relating to the designing and detailing of machinery for movable bridges. This information, together with some added material, developed, through a committee of the American Bridge Company composed of Mr. Dudley, as Chairman, Mr. R. A. Mather, and P. J. Reich, M. Am. Soc. C. E., into a set of notes entitled, "Notes on Designing Machinery for Movable Bridges." These notes laid the foundation for an enlarged copy of nearly 200 pages developed under the supervision of O. E. Hovey, M. Am. Soc. C. E., Assistant Chief Engineer of the American Bridge Company. In his valuable two-volume work on "Movable Bridges," Mr. Hovey acknowledges the assistance and co-operation given by Mr. Dudley in its preparation.

In the death of Mr. Dudley, the Engineering Profession lost one of the most versatile men in the practical design and detail of movable bridges, inasmuch as the last thirty years of his life were intimately associated with the development of this class of structures, from the early types of the simple hand-operated swing bridge to the largest and most varied types of power-operated swing, bascule, and rolling bridges. He was also well versed in the detailing and designing of all types of steel structures and had an exceptionally keen insight into the fundamentals of engineering.

In acknowledging the worth of a member to the Society and the profession, one should not overlook his value to the community. Sterling in character, Mr. Dudley was of gentle and kindly disposition. He was a devoted husband and father and is survived by his widow, Clara Garvey Dudley, and a daughter, Ruth Dudley Daniells.

Mr. Dudley took an active part in the community life and devoted much time to the advancement of things worth while. He was a delightful companion, and much loved by his fellow citizens and co-workers.

Mr. Dudley was elected a Member of the American Society of Civil Engineers on October 10, 1916.

LOUIS HYDE EVANS, M. Am. Soc. C. E.*

Died June 5, 1929.

Louis Hyde Evans, the son of Enoch W. and Caroline Hyde Evans, was born at Kenosha, Wis., on January 21, 1851. He was the second oldest of a family of four children, two sons and two daughters. His father was a prominent attorney in Kenosha, and Chicago, Ill., to which city the family moved during the Sixties.

After preparation in the public schools of Chicago, Mr. Evans commenced his railroad work, in 1871, first as Rodman, then as Instrumentman, with the Chicago and Northwestern Railway Company on the location and construction of its lines in the Northern Peninsula of Michigan. In the fall of 1872 he resigned to enter Rensselaer Polytechnic Institute, at Troy, N. Y., in the Class of 1876. He left the Institute, however, after six months, and entered the University of Michigan at Ann Arbor, Mich., as a special student in Engineering in the Class of 1875. In July, 1873, he obtained a leave of absence from the University to accept a position with the Erie Railroad Company as Assistant Engineer on the construction of double track on the Delaware Division of that road, grading and bridging 25 miles between Lackawaxen, Pa., and Calicoon, N. Y. He then returned to the University at Ann Arbor in the fall of 1874.

Following his graduation Mr. Evans was again employed with the Chicago and Northwestern Railway Company, and from June, 1876, to 1879, was Assistant Engineer on location and construction of its lines in Minnesota, on the completion of which, in 1879, he was employed on the Central Branch of the Union Pacific Railroad. In the latter part of 1879 he again returned to the Chicago and Northwestern Railway Company and served as Division Engineer and Engineer of Track Elevation in Chicago until 1900, when he resigned to accept the position of Assistant Chief Engineer with the Delaware, Lackawanna and Western Railroad Company at Hoboken, N. J. About 1904 he resigned that position to become a member of a firm of contractors who were making grade reductions on the Delaware, Lackawanna and Western Railroad in Northern New Jersey; after the completion of this work he returned to Chicago as Consulting Engineer for the Union Stock Yards and Transit Company, on its track elevation.

When this work was finished, Mr. Evans became associated with the late Horace G. Burt, Civil Engineer, formerly Chief Engineer and Vice-President of the Chicago and Northwestern Railway Company, and President of the Union Pacific Railroad Company, as a Consulting Engineer on the Chicago Smoke Abatement and Electrification Commission. He then became Consulting Engineer for the Southern and Frisco Railway Systems on the construction of the Chalmette Slip, a deep waterway and terminal at New Orleans, La. On the completion of this work in 1915 he retired from active practice, and devoted the remainder of his life to his personal interests in Chicago, and his country home on the Delaware River, in Pennsylvania.

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* Memoir prepared by a Joint Committee of the Society and the Western Society of Engineers, consisting of Lincoln Bush, Past-President, Am. Soc. C. E., E. C. Carter, M. Am. Soc. C. E., and J. S. Robinson, M. W. S. E.

Mr. Evans was a member of the Western Society of Engineers and of the Zeta Psi Fraternity.

He was married on June 2, 1879, to Frances Helen Robinson, at Mast Hope, Pa., and is survived by his widow and two children, Mrs. E. T. Hiscox, of Tyringham, Mass., and Major E. Webster Evans, of Detroit, Mich.

Mr. and Mrs. Evans celebrated their Golden Wedding Anniversary on Sunday, June 2, 1929, at Mast Hope. Mr. Evans was apparently in fine health at that time, but a sudden heart attack in his sleep, early on Wednesday morning, June 5, caused his death.

His sterling character, his love of justice to all, his enthusiasm in his work, and his encouragement to others under trying circumstances, endeared him to the many who worked with him, or came in contact with him.

Mr. Evans was elected a Member of the American Society of Civil Engineers on July 3, 1889.

JAMES EDWARD FULTON, M. Am. Soc. C. E.*

Died December 6, 1928.

James Edward Fulton, the son of the late Hon. James Fulton, who represented the Taieri, New Zealand, Constituency in the House of Representatives many years prior to his elevation to the Legislative Council, was born at West Taieri, Otago, New Zealand, on December 11, 1854. His preliminary training was received in the District School at West Taieri which, together with intervals spent under private tutelage, completed his education.

When Mr. Fulton was still a very young man, he showed a special aptitude for engineering and in the early Seventies had sole charge of the machinery of a large flax mill for a period of nine months.

In January, 1874, he was articled to Mr. John Carruthers, of Wellington, who was then the New Zealand Government Engineer-in-Chief in the Colony in charge of railways. After Mr. Fulton had served for four years as a Cadet, he passed the Junior and Senior Civil Service examinations, and also one required for the diploma of New Zealand Authorized and Licensed Surveyor. He was then appointed an Assistant Engineer of the Public Works Department, in principal charge of some railway location through heavily forested and rough country. He also served as First Assistant on the construction of 60 miles of railways, permanent way, and bridges, and had charge of the location and survey of 80 miles of additional railways.

In November, 1880, he resigned his position in the Government service to engage in private practice, but before doing so he was sent by the Department to the Bay of Islands, where he took soundings of the Harbor of Auckland and made a re-survey of part of the railway reserve lands there. During the period that Mr. Fulton was in private practice he went to Hawkes Bay to make some city re-surveys and, in 1882, to Auckland where he made a preliminary survey and estimate of the cost of the proposed line of the Kaihu Valley Railway Company.

From August, 1882, to 1897, Mr. Fulton was connected with the Wellington and Manawatu Railway Company, first as Resident Engineer under H. P. Higginson, Chief Engineer, in charge of the construction of the Palmerston-Waikanae Section. As Referee he also reviewed the various plans for the water supply and drainage of Palmerston North. In August, 1889, Mr. Fulton was promoted to the positions of Manager and Locomotive Superintendent to succeed his brother, the late A. R. W. Fulton. Mr. Fulton took a keen interest in his profession and was ever on the alert to give his Company the benefit of his inventions and improvements. It was while he was in the employ of the Wellington and Manawatu Railway Company that he invented and used in his office a telephone switchboard which was an attractive piece of mechanism, simple in manipulation, although of intricate appearance. He was associated with the movement to introduce from America the first compound locomotives that were used in New Zealand and that are still running with high efficiency on the Manawatu Railway.

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* Memoir prepared by M. Crompton-Smith, Esq., Wellington, New Zealand.

Mr. Fulton retired from service with the Railway Company in 1897 and again entered private practice, undertaking the work of designing and building the Kelburne Cable Tramway in Wellington, which consisted of tunnels, viaducts, and retaining walls, amounting in value to more than £30 000. He also designed a great many bridges for County Councils and the New Zealand Government, and was the Designer and Engineer of a light railway for the Taupo-Totara Timber Company, 50 miles in length, which includes the longest wooden arch span (234 ft.) of any bridge in New Zealand. He served as Engineer for the construction of the railway for the Tongariro Timber Company.

Mr. Fulton's death occurred on December 6, 1928, after almost a year of failing health. As a man of high attainments in his profession and of estimable qualities in other respects, his passing was greatly regretted among a wide circle of friends, professional and otherwise, throughout the Dominion.

He was a member of the Institution of Civil Engineers of London, England, and of its Advisory Committee in New Zealand. He was also a member of the New Zealand Institute of Surveyors, of which he had served as President.

He is survived by his widow, Mrs. C. F. Fulton, of Wellington, and one daughter, who is the wife of Professor Horton, of Holloway College, in England.

Mr. Fulton was elected a Member of the American Society of Civil Engineers on October 4, 1910.

DANIEL WINGERD GROSS, M. Am. Soc. C. E.*

Died January 23, 1930.

Daniel Wingerd Gross was born at Harrisburg, Pa., on September 25, 1871. He was the son of George A. Gross and Mary A. (Wingerd) Gross. Mr. Gross received his technical education at Pennsylvania State College and was graduated therefrom with the degree of Bachelor of Science in Civil Engineering in 1893.

He began work in 1893 as Rodman for the Beech Creek Railroad Company and his professional services before entering the employ of the Atlantic Coast Line Railroad Company were, as follows: In 1893, Rodman, the Beech Creek Railroad Company; 1894, Rodman, the Clearfield, Conemaugh and Western Railroad Company; 1895 to 1899, drafting and designing, Frog, Switch and Signal Department, the Pennsylvania Steel Company; 1899 to 1901, Division Engineer on construction and maintenance, the International Traction Company; 1901 to 1902, Chief Engineer, the Lake Shore Electric Railway Company; 1902 to 1904, Division Engineer on the construction of the Ballston Line, Schenectady Railway Company; 1904, Plant and Mine Engineer for Queens Run Fire Brick Company, Lock Haven, Pa.; 1905 to 1906, with J. G. White and Company, designing special work for electrification of the Sociedad Commercial Lines of Montevideo, Uruguay.

In March, 1906, Mr. Gross went to the Atlantic Coast Line Railroad Company as Engineer in charge of maintenance of way for its Florida Lines, with an office at Jacksonville, Fla. These lines had been acquired by purchase and merger a few years before and, consequently, there was need for definite constructive work by the engineer in charge of maintenance in order that they might conform to the standards of the Atlantic Coast Line System. So excellent were the plans of Mr. Gross for the improvement and so good was the work accomplished that he was soon promoted to the position of Engineer of Construction for the entire System, which position was created for him; his office was established at Wilmington, N. C. During the period in which he held this position the capital account expenditure for extension and improvement of the System for roadway, track, and structures aggregated more than $15 000 000.

When the Act of Congress was passed in 1913 to cover the valuation of the railways in the United States, Mr. Gross was put in charge of the work for the Atlantic Coast Line System and some of its subsidiaries, first, with the title of Chairman of Valuation Committee and, later, as Valuation Engineer. In this capacity, he directed the work of the valuation on the System until the period of Federal Control of railways, in August, 1918, when he was appointed Corporate Engineer. When the railways were released from Federal Control, Mr. Gross was promoted to the newly created position of Assistant to the President. In addition to the work then required of him as Executive he continued to give advice and attention to many of the valuation matters and was serving at the time of his death as a member of the Engineering Committee for the Southern Group of Railways in the Presidents' Conference Committee.

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* Memoir prepared by J. E. Willoughby, M. Am. Soc. C. E.

The work which Mr. Gross performed for the Atlantic Coast Line System during the twenty-four years of his service was of definite value to the property. By that work he established his reputation as a careful, intelligent engineer with such vision for the future as makes for economy. His personality was of a kind to win for himself a host of friends both in and out of railway service.

He died on January 23, 1930, and is survived by three sisters, Miss Helen Gross, of Wilmington, N. C., Mrs. J. M. T. Finney, of Baltimore, Md., and Mrs. John L. Yates, of Milwaukee, Wis. He never was married.

Mr. Gross was elected an Associate Member of the American Society of Civil Engineers on May 1, 1901, and a Member on June 24, 1916.

RICHARD AUGUSTUS HALE, M. Am. Soc. C. E.*

Died December 17, 1928.

Richard Augustus Hale, the son of Bernice Sargent Hale and Sophia Kidder Hale was born on December 3, 1852, at Lowell, Mass.

After graduating from the Lowell High School, Mr. Hale entered the office of the late Hiram F. Mills, Hon. M. Am. Soc. C. E., Hydraulic Engineer, in Boston, Mass. A few months later Mr. Mills was appointed Engineer of the Essex Company in Lawrence, Mass. Young Hale went with him as his Assistant and remained in this office until he entered the Massachusetts Institute of Technology in 1873.

He was graduated from the Massachusetts Institute of Technology in 1877 with the degree of Bachelor of Science in the Department of Civil Engineering. During his four-year course at "Tech", Mr. Hale continued to work during the summer months with the Essex Company, and after receiving his degree he returned to that Company with which he remained associated continuously. In 1886 Mr. Hale was made Chief Assistant Engineer of the Company; when Mr. Mills died on October 4, 1921, he was made Engineer, in which capacity he served until his death.

Mr. Hale was an outstanding authority on water power and made several hundred reports on water-power sources in New England, New York, and Delaware. He served as Chairman of the Commission appointed by the Court to investigate and define the water rights of various parties at Hinsdale, N. H., and made measurements for the proper sub-division of the rights involved in that controversy. He also served as an expert in many cases of water diversion where hearings were held before commissions and masters.

In 1894 he was appointed a member of the Lawrence Park Commission and served in that capacity for twelve years, being Chairman of the Commission during the latter half of his tenure of office. He was a pronounced golf enthusiast and traveled about the United States, taking part in many tournaments on different courses.

Mr. Hale was a member of numerous clubs and organizations, including the Appalachian Mountain Club, the American Forestry Association, the Boston Society of Civil Engineers, which he served as President in 1916, the New England Water Works Association, the National Geographic Society, the American Civic League, the Massachusetts Civic League, the Merrimack Valley Technology Club, the Technology Club of Boston, the Merrimack Valley Country Club, the Lawrence Canoe Club, the Lawrence Boys' Club, and the Monday Night Club.

He was not merely a member of this long list of clubs, but one to whom membership meant service. His constant attendance at meetings of the different societies and his willingness to serve on committees, made him always outstanding. For this and his other winning qualities, Mr. Hale will long be remembered by his engineering associates, his neighbors, and his friends.

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* Memoir prepared by Charles T. Main and Arthur T. Safford, Members, Am. Soc. C. E.

His was the unusual distinction of having worked for only one employer during his entire professional career, and that a long one. This includes not only his undergraduate days, but afterward, a single noteworthy service of more than fifty-one years. Few engineers can boast such a remarkable record.

On October 28, 1880, he was married to Arabella Johnson Plummer. Mrs. Hale, three sons, Frank B., Elliot K., and Richard A., Jr., and two daughters, Helen P. Hale and Mrs. Ernest A. Nordon, survive him.

Mr. Hale was elected a Junior of the American Society of Civil Engineers on February 6, 1884, and a Member on July 1, 1891.

ROBERT JOHN HARDING, M. Am. Soc. C. E.*

DIED JULY 31, 1929.

Robert John Harding, the son of Albert G. and Mary A. (Peck) Harding, was born on February 16, 1880, at Allegan, Mich. Soon afterward, however, his people moved to Chatham, N. Y., and this town was his early home.

Mr. Harding was graduated from Cornell University at Ithaca, N. Y., in the Class of 1903. After serving as an Instructor at the University of Pennsylvania, and as Recorder on the United States Lake Survey, he became Superintendent of Public Works in Hudson, N. Y., in 1905. In 1908, he accepted a similar position in the larger city of Poughkeepsie, N. Y., where he rendered distinguished service until 1912. During the latter year he was engaged in private practice in Poughkeepsie.

In 1913, he went to San Antonio, Tex., as Manager of the Water Supply System which was owned by Belgian interests. He managed the extensive property for the foreign owners with ability and fidelity during the troublesome years of the World War.

After the war, the Belgian stockholders desired to realize on their investment and Mr. Harding formed a group among neighbors to purchase the stock and take over the property. As Chief Engineer and Vice-President, he continued to manage and extend the property, known as the San Antonio Water Supply Company, until it was sold to the City of San Antonio in 1925.

After the sale of the water-works, Mr. Harding was engaged principally in the development of several small water companies, but he made a professional trip to South America and, later, assisted in the investigation of sewers in the Borough of Queens in the City of New York.

Mr. Harding was interested in Red Cross work and was Chairman of the San Antonio Committee during the war period. He had a keen insight and was rarely wrong in his judgment of the men and conditions with which he had to contend. He was quiet and unassuming, but forceful withal—a competent executive and a loyal friend.

He was fond of hunting and fishing and was a most delightful companion on camping expeditions in search of wild ducks and turkeys. One of the writer's most cherished memories is of a long automobile trip from Texas to Arizona, with visits to all the most important dams along the route.

Mr. Harding was a member of the American Water Works Association, the New England Water Works Association, and the American Public Health Association. In the prime of his active manhood, and with much to look forward to, he was killed in an automobile accident for which he was in no way responsible.

He was married on December 25, 1903, to Louise E. Swick, of Mecklenburgh, N. Y., who survives him, with three sons, two daughters, and one grandchild.

Mr. Harding was elected an Associate Member of the American Society of Civil Engineers on November 4, 1908, and a Member on September 3, 1912.

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* Memoir prepared by the late Allen Hazen, M. Am. Soc. C. E.

WILLIAM WALLACE HAYDEN, M. Am. Soc. C. E.*

Died January 7, 1929.

William Wallace Hayden, the son of James Francis and Catherine (McKittrick) Hayden, was born on August 7, 1861, at Baton Rouge, La.

Mr. Hayden's professional career began at a very early age, as he served as a Rodman in 1877 and 1878 on the New Orleans, Texas and Pacific Railroad (now the Texas and Pacific Railway). In 1879 and 1880, he was an Assistant Engineer on the location and construction of the Southern Pacific Extension from Lafayette to Cheneyville, La., and, in 1880 and 1881, he was engaged as Assistant Engineer on the Louisville and Nashville Railroad, from New Orleans, La., to Montgomery, Ala.

At various periods during 1877, 1879, and 1880, he pursued studies at the Louisiana State University in Baton Rouge. From 1882 to 1885, Mr. Hayden was in the employ of the Yazoo and Mississippi Valley Railroad Company (now a part of the Illinois Central Lines), which was then known by other names. During this period he was Transitman on surveys and Resident Engineer in charge of bridging and track-laying. In 1886, he served as Division Engineer on the Burlington and Missouri River Railroad for a time and subsequently was employed by the Kansas City, Memphis and Birmingham Railroad Company, first on location and then as Resident Engineer.

Mr. Hayden then returned to the Yazoo and Mississippi Valley Railroad and the Illinois Central Railroad Companies with which he remained for many years. He resurveyed the Yazoo and Mississippi Valley Railroad from Memphis, Tenn., to New Orleans; located and built the Bayou Sara Branch; and also located many other lines, including that from Centreville to Woodville, Miss., and one from Wilczinski, to Dahomey, Miss. The lines from Jackson to Ethel, La., and from Jackson to McManus, La., and the line from Minter City to Parsons, Miss., were also included in his construction work. He had charge of all construction from the Ohio River southward on the Illinois Central Railroad and from Memphis south on the Yazoo and Mississippi Valley Railroad until about 1905.

He was later appointed Chief Engineer and afterward Assistant to the President of the Mobile, Jackson and Kansas City Railroad Company, after which he entered the employ of the Missouri Pacific Railroad Company, locating the Hickman Branch. In 1906 and 1907, he was Locating Engineer and then Principal Assistant Engineer on the New Orleans, Great Northern Railroad. He next located and built the Lakeview Traction Company Line extending out of Memphis, and was subsequently Chief Engineer of the proposed Memphis and Pensacola Railroad. He was afterward employed by the New Orleans and Northeastern Railroad Company on maintenance of way. After a brief period, however, he left that Company to serve as Pilot Engineer, first on the Gulf Coast Lines and, later, on the Missouri, Kansas and Texas Lines.

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* Memoir prepared by J. F. Coleman, President, Am. Soc. C. E.

Mr. Hayden's health could no longer stand the strain of such an active life. In consequence, he forsook the railways which he had served long and faithfully and in 1924 accepted employment with the Louisiana State Highway Commission. He remained in this position until shortly before his death, which was caused by a severe heart attack.

He was a most genial man who made and kept many friends. He had a keen sense of humor and was ever ready for a joke. He was active and energetic, and devoted to the work of railway location in which he was especially proficient. Indoor occupations were irksome to him, and many of his changes of business were made primarily to escape work which kept him indoors.

He was married on December 16, 1891, to Ruth Davenport, of Port Gibson, Miss., who survives him. He also leaves two daughters, Mrs. C. R. Bostick, of Wauchula, Fla., and Mrs. Talmadge Kinnebrew, of Homer, La.

Mr. Hayden was elected a Member of the American Society of Civil Engineers on November 6, 1895.

HARRY FOOTE HODGES, M. Am. Soc. C. E.*

Died September 24, 1929.

Harry Foote Hodges was born in Boston, Mass., on February 25, 1860, the son of Edward Fuller and Anne Frances (Hammatt) Hodges. Both his parents belonged to families well known and respected. His father, Edward Fuller Hodges, was one of the most prominent and successful members of the Suffolk County Bar. Harry Foote Hodges traced his descent directly from William Hodges, the founder of the Taunton Branch of the family, who emigrated from England to Taunton, Mass. and died there in 1654. Others of his paternal grandparents were George Hodges who served with distinction in the French and Indian War, and Silas Hodges, a Surgeon in the Continental Army and many times in personal attendance on General Washington.

As a boy, Harry Foote Hodges prepared for college at the Boston Latin School and the Adams Academy, at Quincy, Mass. He entered the United States Military Academy at West Point, N. Y., in 1877, and as a cadet he early began to show the qualities which were later to distinguish him. One of the youngest members of the graduating class of 1881, he stood fourth in the Class. As a First Classman he was Cadet Lieutenant and Quartermaster. On being graduated, he received a commission as Additional Second Lieutenant in the Corps of Engineers of the Army.

After serving for four years at the Engineer School and as Adjutant of the Battalion of Engineers at Willets Point, N. Y., Lieutenant Hodges was assigned as Assistant to the District Engineer in charge of works on the Great Lakes, with headquarters at Detroit, Mich. He remained on this duty until 1888, principally engaged on the design of parts for the Poe Lock at the Sault Ste. Marie Canal, for which he also designed the steel lock-gates.

Lieutenant Hodges then returned to the U. S. Military Academy as Assistant Professor of Civil and Military Engineering. After this tour of duty he was appointed as Assistant to the late Col. Amos Stickney, Corps of Engineers, U. S. A., at Cincinnati, Ohio, on the improvement of the Ohio and tributary rivers. In 1893, he was promoted to the grade of Captain and placed in charge of improvements of the Upper Missouri and other rivers. This work included dike construction, snagging, and extensive surveys. On his relief from this detail, Captain Hodges served as a member of the Engineer Board, and originated the design of many features for mounting modern sea-coast guns.

At the outbreak of the Spanish War, he was appointed Lieutenant-Colonel and then Colonel of the 1st U. S. Volunteer Engineers, Porto Rican Expedition. During the war he remained in Porto Rico making roads and surveys and constructing defensive works, reservoirs, refrigerating plants, and bridges. Returning to the United States in 1899, he was placed in charge of river improvements at Cincinnati, until 1901, when he became Chief Engineer under General Wood, in the Department of Cuba, in charge of the works of the Port of Havana and other military and civil projects. On his return from Cuba, he was assigned to the Office of the Chief of Engineers in Washington, D. C. He was promoted to the grade of Major shortly after surrendering his commission as a Colonel of Volunteers.

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* Memoir prepared by Duncan Hodges, Esq., Lake Forest, Ill.

In 1907, while Major Hodges was still on duty in the Office of the Chief of Engineers, the late George W. Goethals, Major-General, U. S. A., M. Am. Soc. C. E.—who had been appointed to succeed John F. Stevens, Past-President Am. Soc. C. E., as Chief Engineer of the Panama Canal—expressed his desire to the President to have an officer of the Canal Commission who was thoroughly familiar with lock structures and competent to design them. He suggested Major Hodges, explaining to the President his connection with the Sault Ste. Marie Canal and the design of the gates for the Poe Lock. However, the late Gen. Alexander MacKenzie, U. S. A. (*Retired*), Hon. M. Am. Soc. C. E., then Chief of Engineers, objected to the relief of Major Hodges on the ground that his departure would cripple the work of the Department. Shortly after, however, Major Hodges was assigned as General Purchasing Officer for the Isthmian Canal Commission, with headquarters in Washington. As such he supervised the purchase of immense quantities of materials required for the construction of the Panama Canal.

In August, 1907, President Roosevelt appointed Lieutenant-Colonel Hodges a member of the Panama Canal Commission. On the Commission he ranked next to General (then Colonel) Goethals, and was Acting Chairman in the latter's absences. As Assistant Chief Engineer of Canal Construction, he was given general supervision of the design and construction of the locks, dams, and control works. A number of the most valuable features of the Canal are the direct results of his engineering skill.

In Joseph Bucklin Bishop's book entitled "The Panama Gateway", there appears the following paragraph in which General Goethals estimates Colonel Hodges' services:

> "He took over the designing work for the Panama Canal at a time when definite plans had to be adopted and the work carried to completion. Comprised in that work were designs for the dams, locks, gates, spillways, valves, operating machinery, hydro-electric station, and aids to navigation. He was placed in charge subsequently of the erection of the gates and the installation of valves and operating machinery. Charged with the solution of the most important engineering problems of the canal, it can be said of him truthfully *that the canal could not have been built without him.*"

It must not be supposed that Colonel Hodges completed the design and construction of the locks and dams of the Panama Canal without much technical criticism by engineers in the United States. During construction days on the Canal, it was often heard that certain of the designs of the locks were faulty. It was even rumored that the methods of suspending the 730-ton lock-gates were inadequate to the loads imposed. In view of the magnitude and uniqueness of the task assigned to Colonel Hodges, it is not surprising that such criticism existed. Nearly eighteen years have passed since the first gate was swung into position. Constant operation of the lock machinery without essential modification has proved its soundness beyond all doubt. The fact that able engineers at one time criticized the design now shows how difficult the project was technically, and how great the feat accomplished.

Shortly after the first steamer passed through the Canal on August 15, 1914, Colonel Hodges returned to the United States. In March, 1915, he was promoted to the grade of Brigadier General of the Line of the Army and received the distinguished honor of the Thanks of Congress for his services. He remained in Washington in charge of the Washington Engineer District until May, 1915, when he assumed command of the North Atlantic Coast Artillery District, with headquarters at Fort Totten, N. Y.

In August, 1917, promoted to the grade of Major General (National Army), he assumed command of the 76th Division, Camp Devens, Massachusetts. The supervision of the building of the Cantonment and the organization and training of the Division were entrusted to him. On December 10, 1917, he was ordered overseas on an observation tour of the Allied Forces in France, returning in February, 1918. While in France, he was with the New Zealand Division, 2d British Army, in the Ypres Sector, and the 4th Division, French 2d Army, in the Avecourt Sector. On returning to the United States he again took command of the 76th Division. In July, 1918, he sailed with his Division for France, where he remained until after the signing of the Armistice. Among his awards was the Distinguished Service Medal, with the following citation:

> "For exceptionally meritorious and distinguished services in a position of great responsibility. As Commanding General, Camp Devens, Massachusetts, he displayed unusual administrative and executive ability, sound judgment and high professional skill. He established a model system of schools and training, organized and trained the 76th Division, and in addition thereto trained for overseas service more than 40 000 men of other units. His untiring energy, devotion to duty, coupled with other outstanding soldierly qualities, contributed markedly to the successful operations of the American Forces during the World War."

After the war, General Hodges commanded, successively, the 20th Division at Camp Sevier, South Carolina, Camp Travis, Texas, and the North Pacific Coast Artillery District, retiring in the grade of Major General on December 22, 1921, after more than forty years' continuous and conspicuous duty in the service of his country.

General Hodges possessed, to an unusual degree, all the qualities of the successful engineer, combining with high mental alertness a great power of concentration. He was able to deal successfully with a multitude of details without losing sight of the end to which he strived. He prepared himself painstakingly for each task assigned him. One of his foremost characteristics was his mental integrity; he could not bring himself to countenance either carelessness or inaccuracy. A man of such energy and driving power, he required his subordinates to live up to the standard he set for himself. He contrived to accomplish this without nagging or undue pressure. Quiet and self-possessed, he never resorted to the tactics of the slave driver. By his own example he was able to gain and keep the respect and loyal support of those under him. In later years, when he held positions of high command, he never permitted himself to hide behind his rank; he faced all issues squarely as they were presented to him. He was exceedingly astute in grasping the details of plans submitted to him. In whatever capacity, or wherever he served, men soon learned that he quickly got to the bottom of any problem and, having determined all the facts to his satisfaction, took action promptly and efficiently.

After his retirement from active service, General Hodges made his home in Lake Forest, Ill. He devoted his time to reading, studying, and his favorite outdoor sports, shooting and golf. In September, 1926, he suffered the loss of his wife, Alma L'Hommedieu Reynolds, to whom he was married in December, 1887. The attachment between the two was such that it could not be supposed that he would long survive her. General Hodges died at his home in Lake Forest on September 24, 1929, and is survived by his daughters, Mrs. A. H. Acher and Mrs. G. L. Dickson, and his son, Duncan Hodges.

General Hodges' death brought many tributes from all parts of the United States showing the great respect and high regard in which he was held. Gen. Charles P. Summerall, U. S. A., Chief of Staff, wrote in part that:

> "From the date of his appointment as Second Lieutenant in the Corps of Engineers until his retirement as a Major General more than forty years later, General Hodges had undertaken the performance of every duty assigned him with characteristic zeal and energy. His successful accomplishments were only limited by the number and variety of the responsibilities entrusted to him. In his death, which is deeply regretted, the Army loses one of its most brilliant and successful officers."

General Hodges was elected a Member of the American Society of Civil Engineers on March 4, 1903.

CAREY VANDERVORT HODGSON, M. Am. Soc. C. E.*

Died May 19, 1929.

Carey Vandervort Hodgson, the son of Lorenzo and Clara Emma (Hyatt) Hodgson, was born on July 11, 1880, at Wilmington, Ohio. He was educated in the public schools, at Wilmington College, and at Haverford College, Haverford, Pa. He received the degree of Bachelor of Science from Wilmington College in 1902, and from Haverford College in 1903.

On March 31, 1904, he entered the field force of the United States Coast and Geodetic Survey with which he remained, except for two short periods, until his death. He was successively promoted to Assistant and then to Hydrographic and Geodetic Engineer, with the relative rank of Lieutenant Commander in the Navy (May 18, 1920). From December 18, 1905, to October 20, 1906, he was in commercial employment, and during the World War he served in the United States Army.

Under Executive Order of September 24, 1917, he was transferred to the Army with the rank of Captain in the Engineers Reserve Corps. His military service included the following: 1918, Topographical Officer (about 4 months) and commanding Company B (about 4 months), 104th Pioneer (sapper) Engineers, United States and France; Topographical Officer, 29th Division, and Assistant Division Engineer, France (more than 1 month); October, 1918, became Major, and for about 4 months commanded 2d Battalion 104th Engineers, France. By Executive Order of February 26, 1919, he returned to duty in the Coast and Geodetic Survey, reporting on March 19 of that year.

Major Hodgson had a very wide experience in charting and mapping work with the United States Coast and Geodetic Survey, during the twenty-five years he was connected with that Bureau. When he first entered the Survey he was assigned to one of its vessels, the *Matchless*, engaged on the hydrographic survey of the Potomac River; later, he was assigned to the *Blake*, operating on the New England Coast. He then had a season of topographic work along the Potomac River and, subsequently, returned to ship duty. This work carried him to many points on the Atlantic and Pacific Coasts and along the coasts of Alaska, as well as to the Philippine Islands. During the earlier period of his duty in the Islands, he was in charge of a launch party surveying along the coasts of Bohol, Cebu, Leyte, and Samar. The work included the original survey of the San Juanico Straits. Later, he commanded the Coast and Geodetic Survey Steamer, *Research*, which was engaged on surveys on the southern coast of Masbate.

After serving three years in the Philippines (1908-11), he returned to the United States and was assigned to the determination of telegraphic longitudes in the United States and in Alaska, in co-operation with the late Edwin Smith, M. Am. Soc. C. E., then an Assistant in the Coast and Geodetic Survey.

In 1912, the Director of that Bureau decided that an attempt should be made to complete in a single season an arc of first-order triangulation, running approximately along the 104th Meridian from the vicinity of Pike's Peak,

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* Memoir prepared by William Bowie, M. Am. Soc. C. E.

Colorado, to the Canadian boundary. This triangulation was needed to furnish strong geographic positions at the international border on which the boundary surveys could be based. Major Hodgson was chosen as Chief of one of the two triangulation parties engaged on that work. He had had previous experience on second-order triangulation in the Philippines, and the reports of his superiors showed that he had accomplished the work assigned to him in an efficient manner. It was for this reason that he was selected for work on the 104th Meridian.

Plans were perfected in the Washington Office and the two Chiefs of Party took the field early in the summer. During a period of seven months they completed an arc of triangulation 720 miles in length. This was rapid progress, but the work was done with standard accuracy and at low unit cost.

Major Hodgson's next assignment consisted of first-order triangulation and astronomic determinations of latitudes. In 1914, he was engaged for more than seven months in the determination of such latitude in Texas, New Mexico, Arizona, California, and Nevada. During the season of 1913, he used a 1½-ton automobile truck to transport his party from station to station while on astronomical work. This was the first time that the Coast and Geodetic Survey had used an automobile truck in its surveying operations. During 1912, an engineer of the Bureau had hired a truck for use while measuring some base lines along the 49th Parallel, in connection with the United States-Canada Boundary Survey. That officer had been assigned to the Boundary Commission to carry on base-line operations. The truck proved so successful on the boundary work that it was decided to use it as an experiment on the work that Major Hodgson had under his direction in 1913. The test was so successful that the truck became the standard means of transportation for geodetic surveying parties.

From 1914, until he entered the Army in 1917, he was engaged on various triangulation and traverse projects. On his return to the Survey, at the close of the World War, he was placed in charge of the first-order triangulation party working on what was called the El Reno (Okla.)-Needles (Calif.) arc of triangulation. This arc crosses the States of Oklahoma, Texas, New Mexico, and Arizona. After one season on this work, he was given, in May, 1920, the assignment of Assistant Chief of the Division of Geodesy, on duty at the Washington Office of the Survey. He occupied this position at the time of his death.

While serving as Assistant Chief of the Division of Geodesy, Major Hodgson had direct charge of the field force of the Geodetic Branch of the Survey. He made annual inspections of the geodetic parties in the field and prepared the instructions under which the work was done. He studied carefully the practices of other countries as shown in the official reports received from geodetic organizations abroad. He prepared the copy for two manuals of the United States Coast and Geodetic Survey, "Manual of First-Order Triangulation",* which appeared in 1926, and the other, "A Manual of Second and Third-Order Triangulation and Traverse",† which was issued about the time of his death.

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* *Special Publication, No. 120,* U. S. Coast and Geodetic Survey.

† *Special Publication, No. 145,* U. S. Coast and Geodetic Survey.

While at the Washington Office, Major Hodgson took an active part in engineering matters. In addition to his official duties, he became a member of the American Military Engineers, and the Washington Society of Civil Engineers. He was also a member of the American Geographical Society, the Washington Academy of Sciences, the Philosophical Society of Washington, and a Fellow of the American Association for the Advancement of Science. He was a Director of the Washington Society of Engineers in 1927 and 1928, and also a Director, from 1927 to 1929, of the Society of American Military Engineers. Since March 2, 1914, he had been a member of the Cosmos Club, of Washington, which may be called the scientific headquarters of the city.

On the organization, in December, 1926, of the Division of Surveying and Mapping of the Society and the election of an Executive Committee of that Division, Major Hodgson was appointed Secretary to the Committee. He served in that capacity until his death. He took an intense interest in the affairs of the new Division and helped to organize a number of standing and special committees formed to report on different phases of surveying and mapping. It may well be said that the success of the Division of Surveying and Mapping is due, in a great extent, to the efforts and ability of Major Hodgson. He represented the Chairman of the Division at the meeting of the Committees on Technical Procedure of the Society, held in Dallas, Tex., in April, 1929.

Major Hodgson and his 10-year old son, William Hockett, an only child, were drowned Sunday, May 19, 1929, while canoeing along the shore of Chesapeake Bay, in the vicinity of Annapolis, Md. He and his wife and son were spending the week-end at their cottage at Bay Ridge, about four miles south of Annapolis. He decided to test his canoe under sail during the forenoon of the day of his death. The canoe had been used by him and his son for the last two years, but had never before been equipped with a sail. No one saw the accident, but it is believed that the rather fresh wind carried the canoe from smooth water out into the increasingly rough waters offshore and made return from the capsized craft impossible, against the rising wind and sea.

In the early afternoon a search was made for the canoe and its occupants by an airplane, a submarine chaser, and launches stationed at the Naval Academy at Annapolis. The search was continued until dark and was resumed at daybreak, Monday. No trace whatever was discovered until the canoe was found, the following Wednesday, floating upside down, Major Hodgson's body under it, with the painter of the boat wrapped around his arm several times. His son's body was not recovered until June 2, 1929 (two weeks after the accident). It was found across the Severn River from Annapolis, twenty miles or so from where Major Hodgson and the canoe were found. Major Hodgson was buried, with full military honors, in Arlington National Cemetery, on May 25, and on June 3 the boy's body was buried beside that of his father.

Major Hodgson was married on April 17, 1916, to Edith Hockett, of Westboro, near Wilmington, Ohio, who survives him.

Due to his writings, his activities in engineering societies, and his administrative duties in the Coast and Geodetic Survey, Major Hodgson was recognized as one of the leading engineers of the United States in the field of surveying and mapping. He was considered an outstanding authority in these lines. In addition to his high professional qualifications, he possessed a personality and character that impressed most favorably all with whom he came in contact and endeared him to many. His untimely death is a great loss not only to the Government Service, but to the entire Engineering Profession.

Major Hodgson was elected a Member of the American Society of Civil Engineers on November 25, 1919.

HOWARD KINGSBURY HOLLAND, M. Am. Soc. C. E.*

Died October 29, 1927.

Howard Kingsbury Holland was born in Edwardsburgh, Mich., on March 24, 1884. He was the second son of Dr. Marion Holland and Harriet (Kingsbury) Holland. His father was a physician and druggist and his paternal grandfather, Dr. Samuel Ward Holland, was one of the pioneer physicians of Eastern Michigan. His maternal grandfather was Asa Kingsbury, a banker and merchant of Western Michigan.

In 1894, Mr. Holland's parents moved to Cassopolis, Mich., and he was graduated from the Cassopolis High School in 1901. The following summer he entered the employ of the Page Wire Fence Company, at Monessen, Pa., in which position he was engaged at first in the drafting-room and, later, as a helper on the open-hearth furnace.

In the fall of 1902, Mr. Holland entered the University of Michigan, at Ann Arbor, Mich., but left in the late fall of 1903 to accept an appointment as Committee Clerk in the Senate of the State Legislature of Michigan. In the spring of 1904, he accepted a position with the Reo Motor Car Company, of Lansing, Mich., and remained at this work until the opening of college in the fall of 1905. Mr. Holland was graduated from the Civil Engineering Department of the University of Michigan in June, 1908.

Following his graduation, he was appointed Assistant in the Surveying Camp of the University of Michigan under the late Joseph B. Davis, M. Am. Soc. C. E. After the close of the Camp in the fall of 1908, he accepted a position with Gardner S. Williams, M. Am. Soc. C. E., Consulting Engineer of Ann Arbor, and for the following year was Chief of Party on a survey of approximately 12 000 acres of land for power plants and flowage along the Huron River, near Ann Arbor. From November, 1909, to June, 1910, he was a Resident Engineer for the J. G. White and Company, on the construction of a hydraulic power plant at Sterling, Ill.; and from July, 1910, to September, 1917, he was again associated with Professor Williams, as Resident Engineer and Supervisor of Construction, on a number of water-power and water-works plants.

From July, 1917, until his death, Mr. Holland was a member of the firm of Holland, Ackerman, and Holland, Consulting Engineers, of Ann Arbor, specializing in the design and construction of water-power plants. His last work was in connection with two plants on the Menominee River in Michigan.

Mr. Holland had been prominent in the Masonic orders for a number of years. He was affiliated with the Blue Lodge at Cassopolis, Washtenaw Chapter, Royal Arch Masons, at Ann Arbor, and was a member of the Shrine and a Past Commander of Ann Arbor Commandery No. 13, Knights Templars. He was also a member of the First Congregational Church, of Ann Arbor, of which he was a Deacon at the time of his death.

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* Memoir prepared by Louis E. Ayres, M. Am. Soc. C. E.

He was married on September 24, 1914, to Alma Marie Schmid, of Manchester, Mich. He is survived by his widow and three children, Howard Kingsbury, Jr., Roberta Louise, and John Marion, and by his brother, Ray Kingsbury Holland, Assoc. M. Am. Soc. C. E., all of Ann Arbor.

Mr. Holland was popular as a student and was a member of a number of campus societies, including Michigamua, Tau Beta Pi, and Owls; among his classmates he was highly respected. As a citizen, his counsel was frequently sought, and he took an active part in public affairs. As an Engineer, his principal services were rendered in the field, and all who knew him recognized his thoroughness in construction supervision and his sound judgment in those important problems connected with foundations of dams, power houses, and similar structures. Although of a quiet and retiring disposition, he was possessed of a strong and manly character. He will be greatly missed by many to whom he was a genuine and substantial friend, and his untimely passing is a regrettable loss both to his community and to his profession.

Mr. Holland was elected a Junior of the American Society of Civil Engineers on June 1, 1909; an Associate Member on October 29, 1912; and a Member on March 9, 1920.

RALPH HILLS HOWARD, M. Am. Soc. C. E.*

DIED SEPTEMBER 20, 1928.

Ralph Hills Howard was born in Zanesville, Ohio, on August 15, 1870, the son of Caleb Baxter Howard and Violetta Howard.

After completing his high school education and special study in civil engineering under the late Edmond Turner, M. Am. Soc. C. E., he entered railway service in October, 1889, as Draftsman with the Cincinnati and Muskingum Valley Railway Company. Until April, 1901, Mr. Howard served successively as Draftsman, Assistant on Engineering Corps, and Chief Clerk to the Engineer, Maintenance of Way. He designed and had direct supervision of the construction of masonry for replacing wooden, with steel, bridges, and of the construction of a terminal yard, car repair and machine shops, and an engine house at Lancaster, Ohio.

In April, 1901, Mr. Howard went to Pittsburgh, Pa., as Assistant on the Engineering Corps of the Pennsylvania Lines West of Pittsburgh, where he was engaged on maintenance and construction work. In July, 1902, he was transferred to the St. Louis Division of the Vandalia Railway as Assistant Division Engineer in charge of maintenance and construction. In April, 1905, he accepted service with the Chicago and Eastern Illinois Railroad Company as Principal Assistant Engineer in charge of construction and improvements, and in October of the same year, he was promoted to the position of Engineer, Maintenance of Way, in direct charge of all maintenance and construction work.

From May, 1910, to January, 1911, Mr. Howard was engaged on special work in connection with railroad properties for Eastern capitalists, and from January to June, 1911, he was Chief Engineer of the Great Southern Lumber Company and Engineer, Maintenance of Way, of the New Orleans Great Northern Railway Company in charge of construction and maintenance. In June, 1911, he was appointed General Manager of the New Orleans Great Northern Railway Company, remaining in this position until May, 1915, when he was appointed Chief Engineer, Maintenance of Way, of the Wabash Railway Company, with headquarters in St. Louis, Mo. In October, 1923, Mr. Howard was appointed Chief Engineer of the Company, which position he held at the time of his final illness.

Mr. Howard was a man of sterling character, and of a very pleasant personality, highly regarded by every one with whom he came in contact. He had a host of friends, who loved and admired him, and who will miss him greatly. He was possessed of unusual ability as an executive, and a great capacity for organizing and advancing large projects to a successful conclusion. His sympathetic understanding of the problems of his subordinates enabled him to develop efficient and capable men in his organization. The engineers who were associated with Mr. Howard were indeed fortunate. No matter what circumstances arose, even in the midst of some perplexing construction problem or catastrophe from flood, Mr. Howard always possessed a tranquil mind which made his decisions accurate and effective. He was an indefatigable worker and carried with him into his work a zeal and enthusiasm which was contagious and which permeated his entire organization.

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* Memoir prepared by W. S. Hanley and S. M. Smith, Associate Members, Am. Soc. C. E.

Although Mr. Howard had not been in the best of health for several months prior to his last illness, he had still remained active. It was during his vacation at Petosky, Mich., that he was stricken with cerebral hemorrhage, and although he rallied from the first attack, he passed away on September 20, 1928. He was laid to rest in Woodlawn Cemetery, at Zanesville, Ohio, on September 24, 1928.

Mr. Howard was married on May 18, 1897, to Sina Larzelere, the daughter of William G. Larzelere, an influential and respected citizen of Zanesville, and a veteran of the Civil War. While blessed with no children, Mr. Howard's home life was ideally happy; he was always devoted to his wife and home.

He was a member of the Protestant Episcopal Church, and, as a boy, sang in the vested choir in the church at Zanesville. He was interested in sports and was an enthusiastic golfer, this being practically his only diversion from his work. He was a member of the American Railway Engineering Association, the Glen Echo Country Club, and the Missouri Athletic Club of St. Louis.

Mr. Howard was elected a Member of the American Society of Civil Engineers on July 2, 1913.

WILLIAM DOUGLAS HUDSON, M. Am. Soc. C. E.*

Died February 25, 1929.

William Douglas Hudson was born at Rockbridge, Ill., on April 14, 1880. When he was still a very young child, his family moved to St. Louis, Mo., where he received his early education. In 1907 he was graduated from the University of Wisconsin, Madison, Wis., with a degree of Bachelor of Science in Civil Engineering. Preliminary to receiving his degree he had served as Rodman for James Stewart and Company and as Topographic Recorder and Instrumentman for the United States War Department.

From 1907 to 1909, Mr. Hudson was a Detailer and Designer for the Southwestern Bridge Company, Assistant City Engineer at Joplin, Mo., Office Engineer for the Town of Atoka, Okla., and Estimator and Designer for the Corrugated Bar Company. Even at this early period of his career, the trend of his interest seemed to be toward bridge construction, and during the next seven years (1910-17) he served, first, as Chief Draftsman in the Bridge Department, and then as Assistant Bridge Engineer, for the Missouri Pacific Railroad Company.

At the outbreak of the World War, Mr. Hudson attended the Officers' Training Camp at Fort Sheridan, Illinois, and served overseas with the American Expeditionary Forces, as First Lieutenant in the Third Battalion of the 344th Infantry.

Following the war Mr. Hudson became Principal Assistant Engineer with C. E. Smith and Company, St. Louis, participating in numerous important railroad and bridge design projects. These projects included the American Royal Building, at Kansas City, Mo., and the stock yards structures at Kansas City, and Oklahoma City, Okla.; the plans for the conveyor system, North Market Street Dock, St. Louis; the plant of the Barnickel Company, at Webster Groves, Mo.; and numerous track plans and arrangements in East St. Louis, Ill., in connection with the St. Louis Municipal Bridge.

In 1923, he formed an association with Harland Bartholomew and Associates, later organizing and becoming the active head of the William D. Hudson Company, Civil Engineers. In these capacities Mr. Hudson made engineering investigations and plans, with special reference to railroad, bridge, and harbor work in more than thirty cities of the United States and Canada. In general, these studies were part of a comprehensive city plan, involving wide knowledge and experience, as, for instance, the general railroad plan and grade-crossing elimination study at Toledo, Ohio, which was made the basis of a ten-year program between the city and the railroads, and for which the citizens of Toledo voted $4 000 000 as the city's share of the cost in 1925. In other instances, Mr. Hudson prepared complete designs and specifications for particular engineering projects, such as the Franklin Street Viaduct, in Grand Rapids, Mich., a preliminary estimate of damage for the acquisition of more than $40 000 000 worth of property in Los Angeles, Calif., as a part of the Major Street Plan; and the relocation and grade separation of the main-line tracks of the Southern Pacific Railroad, in San José, Calif.

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* Memoir prepared by Harland Bartholomew, M. Am. Soc. C. E.

Mr. Hudson died in St. Luke's Hospital, St. Louis, from pneumonia, on February 25, 1929, following an operation which he had undergone three weeks earlier. He is survived by his widow, two daughters, Winnifred and Frances, and a son, George D. Hudson.

Mr. Hudson was a member of the American Railway Association, the Society of American Military Engineers, the Engineers' Club of St. Louis, and the American Legion. He was also a member of Fellowship Lodge, No. 345, A. F. and A. M., in Joplin, Mo., and of the Scottish Rite and Moolah Shrine in St. Louis.

Mr. Hudson was elected a Member of the American Society of Civil Engineers on March 16, 1925.

CHARLES RAYMOND HUMPHREYS, M. Am. Soc. C. E.*

DIED JUNE 1, 1929.

Charles Raymond Humphreys was born at Wayside, Md., on July 14, 1881. He was the son of Charles Humphreys and Elizabeth (Hungerford) Humphreys, and a descendant of Daniel Humphreys, who came to the United States from England in 1682, and settled in Sussex County, Pennsylvania. His great-great-grandfather, Joshua Humphreys, designed and built the first fleet of American warships, including the frigate, *Constitution* (*Old Ironsides*), and was Chief Naval Constructor during the administrations of Presidents Washington and Adams.

Mr. Humphreys moved, with the family, to Wilmington, N. C., in 1893. His father was a Government Engineer and was transferred to this District at that time. He resumed his primary studies here with Professor Catlett's private classes, later going to Pantops Academy at Charlottesville, Va., where he completed his preparatory schooling. He then entered Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated with the degree of Civil Engineer in June, 1904. He was elected to membership in the Chi Phi Fraternity.

His first engineering work was during the summer vacations of 1902 and 1903, as Instrumentman with the United States Government at League Island Navy Yard at Philadelphia, Pa. After his graduation in June, 1904, and until January, 1905, he was with the Government forces on surveys and soundings of the Mississippi and Ohio Rivers.

From January, 1905, to January, 1907, Mr. Humphreys was connected with the Carolina Trucking and Development Company of Wilmington, in charge of surveys and the location of farm colonies in several Eastern Carolina Counties, and, later, he was Chief of Party on surveys and reports on water-power projects on the Pee Dee River. From January, 1907, to July, 1908, he was with the Waccamaw Land and Lumber Corporation, Bolton, N. C., engaged on title surveys, and railroad location and construction. From July, 1908, to May, 1911, he was City Engineer of Wilmington, and during the time he held this office he assisted in making an appraisal of the privately owned water and sewerage system, which the City bought and to which, under his supervision, extensive additions were made. While serving in this office he also co-operated with the North Carolina Geological and Economic Survey in making investigations and reports on the feasibility of developing the vast areas of overflowed and waste lands in the Coastal Plane Section of Eastern Carolina.

From May, 1911, to May, 1917, Mr. Humphreys conducted a general engineering business, with an office at Wilmington. During this time he established a broad field of practice, requiring the assistance of a number of engineers to carry on his work. He was a good organizer and directed parties in the different branches of the field. He was for a number of years an active member of the North Carolina Drainage Association, the object of which was the promotion and advancement of the reclamation of over-flowed and waste lands for agricultural purposes. He was largely instrumental in the organization of many of the most important drainage districts in the State, directing the surveying and mapping of the areas affected and designing systems of drainage canals that made it possible to return to agriculture many thousands of acres of erstwhile jungle lands, bettering health conditions, as well as enriching those sections which had heretofore been considered worthless, and a menace to good health. He also made surveys and reports on forestry conditions for a number of Eastern North Carolina Counties, and, again, in 1917, he was Chief of Party on investigations and surveys for the development of water power on the Pee Dee River, in the Piedmont Section of North Carolina, for the Carolina Light and Power Company. It was while he was engaged in this work that his career as a brilliant engineer and a useful citizen was interrupted by his entry into the service of his country in the World War with Germany.

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* Memoir prepared by W. K. Allen, Civ. Engr., and Col. R. S. McClelland, Wilmington, N. C.

Immediately after war was declared Mr. Humphreys offered his services, and was ordered to the Officers' Training Camps at Chattanooga, Tenn., and American University, Washington, D. C., for military training, being commissioned with the rank of Captain, Engineer Corps, on June 19, 1917. Captain Humphreys was ordered to duty with the 105th Engineers at Camp Sevier, South Carolina, where he served as Topographical Officer and in various other capacities, leaving the United States with the American Expeditionary Forces on May 27, 1918. While in France, Captain Humphreys acted as Water Supply Officer of the American 2d Corps, and had charge of the survey of the St. Quentin Canal and Tunnel from November 4 to 11, 1918. He participated in the following engagements: Ypres-Lys Offensive, August 19-September 5, 1918; and Somme Offensive, September 20-October 8, 1918, Defensive Sector. Captain Humphreys returned to the United States on May 20, 1919, and was honorably discharged at his request on June 5, 1919.

Immediately after his discharge, Captain Humphreys took an active part in securing options, examining titles, and mapping outlines of a proposed 75 000-acre Soldier-Sailor Home Colony, in Eastern North Carolina, a project then contemplated by the Government, to provide homes and employment to *ex*-service men. He devoted much of his time to this movement during the summer of 1919.

From September, 1919, to September, 1921, he was City Engineer of Whiteville, N. C., during which time he directed a two-year street-paving program and water and sewerage extensions. In October, 1921, he resumed practice as a private engineer, with offices at Wilmington, and continued in this field until his health failed in the spring of 1928. During this time Captain Humphreys was in charge of road work for a number of counties in which modern roads were being built; he wrote plans and specifications and supervised the construction of this work and had a broad field of practice embracing all branches of the profession. His service and opinion on title work were sought by many corporations and holders of large areas of land over the State. He was again employed by the Waccamaw Land Corporation with which he was engaged in 1907.

Captain Humphreys had a pleasing personality and a genial disposition, which won him friends everywhere, and he was ever ready to assist a fellow man in need and give aid to any worthy cause. He was punctilious in his habits, and painstaking in the performance of his duties.

He was married at Wilmington, on November 22, 1910, to Lillian Kenly, who, with their four children, Charles Raymond, Jr., Lillian Kenly, Mary Elizabeth, and Nancy Clark, as well as his mother and two sisters, survives him.

Captain Humphreys was elected an Associate Member of the American Society of Civil Engineers on June 3, 1915, and a Member on January 16, 1928.

GEORGE MARTIN ALOYSIUS ILG, M. Am. Soc. C. E.*

DIED JUNE 28, 1929.

George Martin Aloysius Ilg was born in Chicago, Ill., on November 26, 1881, the son of John J. and Christine (Schnebelen) Ilg, being one of ten children. Mr. Ilg received his early education in the public schools of Chicago and was graduated from Calumet High School in 1901.

His practical experience in engineering began shortly after his graduation, when he went to work for the American Bridge Company. He served four years with this Company as Shop Inspector, Tracer, Detailer, and Checker on steel work for bridges and buildings.

In 1905 he entered the School of Civil Engineering at the University of Illinois, paying his own way with funds he had saved during the previous four years and with the help of earnings as a Draftsman for the Kenwood Bridge Company during the summer vacations. He completed the full four-year course and was graduated in June, 1909, receiving the degree of Bachelor of Science in Civil Engineering.

After his graduation from college Mr. Ilg began his engineering work with the Chicago, Milwaukee and St. Paul Railroad Company and was soon given charge of the design of concrete and steel bridges and other structures. In 1911, he entered the employ of Holabird and Roche, Architects, as a Designer in the Engineering Department. During the next seven years, he was responsible for a large part of the firm's design work in both reinforced concrete and structural steel buildings.

In 1916, in connection with his regular work, he had occasion to design some long-span steel arches for an armory. He became interested in arch design to the extent of devoting considerable additional study in his spare time to this subject, which resulted in his preparation of a thesis on arches. Based on this, together with other requirements, Mr. Ilg received the degree of Civil Engineer from the University of Illinois in June, 1916.

In May, 1917, he left the employ of Holabird and Roche and entered the First Officers' Training Camp, at Fort Sheridan, Ill. He held a commission in the Reserves as Captain of Engineers, which commission was confirmed after training service at Fort Sheridan, Ill., and at Fort Leavenworth, Kans. On the completion of his training he was put temporarily on the inactive list and, while waiting for a call, entered the employ of the Stone and Webster Corporation, at Philadelphia, Pa., on the structural design of buildings for Hog Island Shipyard. His work here apparently was deemed as important as the more active duty in France, as he was not called for active service, but remained on the Hog Island work for the duration of the World War.

At the end of the war, in the fall of 1918, Mr. Ilg returned to Chicago and became Structural Engineer for Jarvis Hunt, Architect, in charge of design and construction. After two years in this position, he resigned on account of poor health resulting, it was thought, from overwork during the war. His health did not improve, and he suffered a severe nervous breakdown. Seeking outside work, he moved to Pierre, S. Dak., in 1920, where he accepted a position as Assistant Bridge Engineer for the South Dakota Highway Commission.

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* Memoir prepared by F. E. Brown, M. Am. Soc. C. E.

After two years on this work, Mr. Ilg's health was very much improved, and in 1922 he moved to St. Paul, Minn. Here, he served for some time in the City Engineering Department, where he was engaged on the design of a number of bridges, including the Ford Bridge between Minneapolis and St. Paul. Later, he entered private practice as a Civil and Structural Engineer, with headquarters in St. Paul.

In January, 1926, Mr. Ilg returned to Chicago and entered the employ of Smith and Brown, Engineers, Incorporated. He was with this firm until his death.

He will long be remembered and esteemed by his associates for his kindly and genial disposition, his friendliness to others, and his interest in the success of his fellow workers. He enjoyed his profession fully and always took a keen delight in any unusual problem, no matter how difficult or tedious. His singularly alert mind and his fine training and experience made his services especially valuable to his associates.

On June 27, 1910, Mr. Ilg was married to Gretchen Zeit, the daughter of Dr. F. Robert Zeit, of the Medical Faculty of Northwestern University. He is survived by his widow and an only daughter, Marguerite.

He was a Mason, a member of the Triangle Engineering Fraternity, Hiram Sleifer Post of the American Legion, the Forty and Eight Club, and the Western Society of Engineers. He was also a Captain, Engineers Reserve Corps, U. S. Army.

Mr. Ilg was elected a Member of the American Society of Civil Engineers on March 12, 1923.

JOHN DOVE ISAACS, M. Am. Soc. C. E.*

Died April 26, 1929.

John Dove Isaacs was born on October 6, 1848, at Richmond, Va. He was educated at the University of Virginia, and was graduated in the Scientific Course, which in those days was closely associated with the Department of Liberal Arts.

From 1869 to March, 1870, he served as Draftsman with Talbott and Sons, of Richmond. He obtained further practical training from March, 1870, to March, 1874, when he was Prize Apprentice and then Draftsman at the Union Iron Works, Baltimore, Md. From March, 1874, to April, 1875, he worked as Draftsman with Hilles and Jones, Wilmington, Del.

In 1875, Mr. Isaacs went to San Francisco, Calif., and entered the service of the lines which now comprise the Southern Pacific Company, first as Draftsman in the Bridge and Building Department, then as Chief Draftsman and Designer, and, later, Principal Assistant Superintendent of Bridges and Buildings. In 1890, he was Consulting Engineer for the Piedmont Cable Company, Oakland, Calif., and, in 1891, Consulting Engineer of the California Bridge Company on the Petaluma River Draw-Bridge. In the early Nineties, he made a trip to Guatemala on a reconnaissance and preliminary survey for an extension farther into the mountains of the Ferrocarril Occidental. From 1891 to 1900, he was Second Assistant Engineer of Maintenance of Way for the Southern Pacific Company, and from 1900 to 1905, Assistant Engineer of Maintenance of Way.

During his period of service from 1875 to 1905, Mr. Isaacs was engaged in many interesting engineering projects, often referred to by him later in life. The following serve as examples: The design of the car-ferry steamer, *Solano;* the wharves and slips for the train-ferry service between Port Costa and Benicia, Calif.; the double-deck, side-wheeler ferry steamer, *Piedmont;* the Oakland Mole wharves, slips, and buildings; the Hotel Del Monte; the reconstruction of the Upper Cascade Viaduct which was burned in 1889 and rebuilt in 3½ days (420 ft. long and 100 ft. high); the development and superintendence of the Southern Pacific Company's activities in wood preservation; the design and construction of a masonry dam and tunnel, at Wright, Calif., on the Santa Cruz Branch; and the engineering and construction features of the California Mid-Winter Fair at San Francisco.

On December 1, 1905, Mr. Isaacs was transferred to the Eastern Executive Offices of the Southern Pacific Company and also became Consulting Engineer of the Union Pacific and Southern Pacific Systems, which meant that his engineering influence was exerted on all the Harriman Lines and their subsidiaries. After the dissolution of the Harriman properties in 1913, he was retained as Consulting Engineer of the Southern Pacific Company, with headquarters in New York, N. Y., which position he held until the time of his retirement in 1923.

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* Memoir prepared by C. R. Harding, M. Am. Soc. C. E.

On a bronze tablet recently erected at Stanford University, Palo Alto, Calif., Mr. Isaacs is given credit for his part in the early development of motion pictures. In the late Seventies, Senator Stanford sought Mr. Isaacs' advice to obtain proof to win a bet of $25 with Mr. James W. Keene, that a trotting horse at one period of its stride has all four feet off the earth at once. Mr. Isaacs decided to use a number of cameras set side by side and make an accurate photographic record of the foot movement; he devised a simple electro-magnetic release that successfully recorded a series of photographs and settled the bet in favor of Senator Stanford. The history of the motion picture shows that others who were interested in these experiments, carried them forward, and that these tests really marked the birth of the motion picture.

Mr. Isaacs was generally recognized as one of the able authorities on wood preservation in the United States, having won distinction by his active work in the practical development of the creosoting and Burnettizing processes. He invented, jointly with the late W. G. Curtis, M. Am. Soc. C. E., a portable wood-preserving plant.

The taper rail devised by Mr. Isaacs to connect two abutting rails of different sections, has been widely used. This simple device, well known to all track men, consists of a piece of rail from 8 to 16 ft. long, so forged that the ends correspond respectively to the two sections to be joined, permitting the use of ordinary joint bars without the need of compromise joints.

The rifled pipe, invented jointly by Mr. Isaacs and the late J. B. Speed, Assoc. M. Am. Soc. C. E., in 1905, afforded a temporary solution to a difficult problem. Some of the heavy California oils could not be transported through pipe lines without expensive preheating, on account of the frictional losses in head. Mr. Isaacs and his Assistant conceived the idea of rifling the pipe like a gun bore, to give the oil a whirling motion. Then, when pressure had been applied, water was introduced, which, being somewhat heavier, would be thrown by centrifugal force, due to the whirling motion of the oil, against the pipe surface and stay there as a thin film, thus acting as a lubricant and permitting much easier passage of the oil in long-distance pumping. This use of water as a lubricant appeared to make it impossible for crude oil to stick to the pipe and impede its progress. A rifled pipe line was constructed between Delano and Volcano, Calif., a distance of 31 miles, which later was extended 282 miles farther from Delano, in the Kern River oil fields, to tide-water at Port Costa.

In 1899, Julius Kruttschnitt, then General Manager of the Southern Pacific Company, appointed committees of engineering officers of the Maintenance and Mechanical Departments to prepare a uniform and consistent system of locomotive tonnage rating on all Divisions of the Company's lines. The paper on "Locomotive Tonnage Rating—Southern Pacific Company—Pacific System",* written by the late Mr. Curtis, Engineer, Maintenance of Way, with the able assistance of Mr. Isaacs, is an outgrowth of this work. After the study of detailed line profiles and extensive mathematical analyses, there was developed a theoretical basis for the rating of tonnage to be handled over a Division by a given class of locomotives at the required speed, according to the time tables for trains. In 1909, the first paper was supplemented by another entitled, "The Economic and Efficient Speed of Freight Trains,"* prepared jointly by Mr. Isaacs and his Assistant, E. E. Adams, Assoc. M. Am. Soc. C. E., now Assistant to the President of the Union Pacific System.

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* *Bulletin*, Am. Ry. Eng. and M. of W. Assoc., November, 1900.

Immediately before his appointment as Consulting Engineer, Mr. Isaacs attended a meeting that was called in 1905 to establish standards for way and structures common to all the Harriman Lines. As a result of this meeting, "Common Standards" were adopted, many of which have stood the test of time and are still in force. Such a system of using standards is still in vogue on the lines that then comprised the Harriman Lines, although they are now operated as separate units.

Important structures built while Mr. Isaacs was Consulting Engineer are too numerous to mention, and only a few of the most notable are recorded here. The double-deck, double-track Sacramento Draw-Bridge is believed to be one of the heaviest ever built. Other examples of his work are the Coos Bay Draw-Bridge, 458 ft. long; the reinforcement of the Pecos River Viaduct, 1 516 ft. long and 320 ft. above the river bed, with practically no interruption to traffic; the lift-bridge over the Willamette River, Portland, Ore.; the Berwick Draw-Bridge over the Atchafalaya River, in Louisiana; the Galveston Freight Terminal and Grain Elevator; the marine ways at Alameda, Calif.; the Empalme Shops, in Mexico; and the Pacific Electric Shops and Yards at Torrance, Calif. He also had charge of the centralized plan of inspection established by the Harriman Lines in 1912.

Mr. Isaacs was a Charter Member (No. 145) of the American Railway Engineering Association, and served on the Committee on Buildings (1900); the Committee on Rails (1908 to 1915 and 1918 to 1929), having been Chairman of the Committee in 1916 and 1917. He contributed the following papers: "Locomotive Tonnage Rating—Southern Pacific Company—Pacific System"; "Effect of Physical Characteristics of a Railway upon the Operation of Trains";† and "The Economic and Efficient Speed of Freight Trains".

Mr. Isaacs passed away at the home of his daughter on Russian Hill in San Francisco, where he had been living quietly since his retirement from active service with the Southern Pacific Company on June 1, 1923. During those six years, he was seen but little by his host of friends. All his strength had been exhausted in his forty-eight years of service to his beloved "Southern Pacific", and he was content to rest during the remaining years of his life.

Mr. Isaacs was elected a Member of the American Society of Civil Engineers on March 7, 1894.

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* *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Vol. II, Pt. 2, p. 1328.

† *Loc. cit.*, p. 1311.

WILLIAM DEAN JANNEY, M. Am. Soc. C. E.*

Died October 30, 1925.

William Dean Janney, the son of William Warder and Marion (Dean) Janney, was born in Baltimore, Md. on September 8, 1859, of English and Scotch-Irish ancestry. After his preliminary education had been completed at the Friends School in Baltimore, he attended Lehigh University at Bethlehem, Pa., for two years.

In July, 1881, Mr. Janney entered the employ of the Baltimore and Ohio Railroad Company, and was engaged on preliminary surveys and location. In November of that year, he was appointed Assistant Engineer with the Norfolk and Western Railway Company, later being placed in charge of the masonry construction on 14 miles of line, which included 2 miles of road extending east from Pocahontas, Va. This work involved masonry for 17 bridges, 100 or more culverts, trestle footings, grading trestles, two tunnels, and continuous track-laying.

In April, 1883, he became General Agent for the Lexington Manufacturing Company, but he soon resigned this position to re-enter engineering. During the winter, he attended classes in Freehand and Architectural Drawing at the Maryland Institute of Art and Design, and, in the spring, he entered the employ of the Annapolis and Baltimore Short Line Railroad Company, which he served as Instrumentman and, later, as Chief Engineer. In this capacity, he completed surveys, made up estimates of cost, designed structures, financed the enterprise, and built the major part of the line.

Three years later, however, Mr. Janney resigned this position to return to the Norfolk and Western Railroad Company. He remained in the service of this road and its affiliated interests until 1894. He was placed in charge of both preliminary and location surveys, which included several hundred miles of construction work, with division terminals, yards, and shops, and other railroad construction of a heavy type. One difficult problem of which he had charge was the foundation work for a bridge across the Ohio and Kenova Rivers in West Virginia. During this period, he was also engaged in the construction of office buildings, the planning of town sites, the erection of dwellings, the grading of streets, the design and construction of a sewerage system, etc. This series of undertakings involved the design and construction of work amounting to several millions of dollars.

In 1891, he was appointed President and General Manager of the Wells Branch Coal Company, for the purpose of conducting the initial development of a new field. This position was created for him by the President of the Norfolk and Western Railroad Company, with which road he had so long been associated. On the recommendation of the Chief Engineer he selected the site, consummated the lease, constructed the necessary works, developed the mine, and at length put it into active operation. When he considered that it had fulfilled its purpose he sold it to private individuals.

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* Memoir prepared by the late A. E. Christhilf, M. Am. Soc. C. E.

From 1894 to 1897, Mr. Janney was engaged in private practice at Ceredo, W. Va., where he undertook and completed contracts for several railroads, foundations for buildings, surveys, and general plans for a bridge over the Ohio River at Ashland, Ky. He likewise examined and made reports on timber and coal lands, conducting expert investigation of territory of obscure location and defective description which he often succeeded in retrieving for the owners. When the celebrated "King Land Case" appeared in Court, he acted as leading engineering expert by appointment of counsel for the defense, and aided in the preparation of cases, testifying at the various trials. He finally prepared all the maps used in the presentation of the case before the Supreme Court of the United States in which the decisive victory was gained.

He was employed during 1896 and 1897, with the United States Engineer Department as Assistant Engineer on surveys of the Ohio River from Pittsburgh, Pa., to Marietta, Ohio, to determine locks and dams for slack-water navigation; later, he was in charge of the Drafting Department and became Acting Geologist for a field party. In the fall of 1897, he was engaged as Chief Engineer with the W. M. Ritter Lumber Company of Welch, W. Va., and Columbus, Ohio, in charge of all surveys, laying off of timber cuttings, and the location and construction of saw-mills. He developed railroads and other means of transportation for logging operations, and determined the measurement of obscure and disputed boundary lines.

From December, 1900, to April, 1904, Mr. Janney acted as Chief Engineer of the Northern Railway Company, of Costa Rica, for the United Fruit Company. At this time, he was in charge of the location and construction of railroads, shops (including the selection, purchase, and installation of all machine tools and appliances), station buildings, bridges, and large apartment and office buildings of steel-concrete.

In July, 1904, he became Chief Engineer for the Elk River Coal and Lumber Company, and for the Buffalo Creek and Gauley Railroad Company in connection with the development of about 80 000 acres of coal and timber land in Clay and Nicholas Counties, West Virginia. After he had built the railroad into the property, designed all the structures, and reported on timber holdings and coal deposits, he started the actual development of the mines. Later, he returned to Baltimore to accept a position as General Superintendent of Construction in the building of new reinforced concrete piers for the City.

In 1911, when this work was nearing completion, Mr. Janney was engaged by D. B. Banks, Consulting Engineer, of Baltimore, to take charge of work in connection with a high-pressure system of water supply in West Virginia. In this capacity he made a topographic survey of a stream valley to determine impounds and location of dams, and, later, he located 43 miles of pipe line. He conducted all negotiations for the acquisition of lands, etc.

From 1912 to 1919, he was engaged in private practice with Mr. Banks, making examinations and reporting on various hydro-electric projects, waterworks, and irrigation systems. This work consisted also of reconstruction, involving intricate engineering and construction problems, hydrographic surveys, layout of channel dredging, and the preparation and testifying in many cases of litigation on damage suits.

During the first year of the World War Mr. Janney served as Assistant Engineer with the Quartermaster Corps, U. S. Army, in charge of the distribution and allotment of railroad materials, but soon after, he was forced to resign on account of ill health.

In December, 1918, he was again engaged in private practice, and subsequently until his death acted as Special Engineer with the United Railways and Electric Company of Baltimore.

Mr. Janney was a man of high character and was well liked by all his associates. He was a member of the Engineers Club of Baltimore, and for several years the Editor of the monthly *Journal* for that Society. He was intensely interested in the organization of The Boy Scouts of America, and was a member of the Court of Honor in awarding badges. He was also an Elder and Trustee of the Second Presbyterian Church of Baltimore.

He had few recreations, but took great delight in tinkering around his own home, or in fishing, spending many hours with his rod in hand waiting for the quick jerk which told of an unsuspecting nibble.

He was married in Baltimore on February 16, 1886, to Marion Rowe; they had five children, Marion, William Rowe, Gertrude, John W., and Charles Homer.

Mr. Janney was elected an Associate Member of the American Society of Civil Engineers in 1891, and a Member on April 1, 1896.

HENRY SCUDDER JAUDON, M. Am. Soc. C. E.*

DIED AUGUST 22, 1929.

Henry Scudder Jaudon, the son of W. A. and Mary Jaudon, was born in Savannah, Ga., on July 20, 1871. He was an alumnus of the Georgia School of Technology and Lehigh University, from which institutions he received his degree in Engineering in 1895 and 1896.

Prior to his graduation Mr. Jaudon secured employment, during vacation periods, with firms engaged in engineering projects of importance, and he profited by the contact thus formed. Following his graduation he was made Chief Draftsman and Inspector of the dry dock at Port Royal, S. C., Inspector for the Pottsville, Pa., Iron and Steel Company, and, later, was placed in charge of dredging operations for the United States Government. In 1898, he was employed by the American Water Works and Guarantee Company, of Pittsburgh, Pa., in the service of which he remained for five years, designing and supervising the construction of a number of water-works plants, including those at Birmingham, Ala., Shreveport, La., and Meridian, Miss.

In 1903, he organized the H. S. Jaudon Engineering Company, which operated extensively throughout the South. The efficiency of his organization was well known and brought him a lucrative practice. He pointed with pride to his achievements in more than a score of cities and counties where he had planned and directed municipal improvements and highway work.

As a citizen, Mr. Jaudon was public-spirited in an unusual degree. He took an active interest in matters pertaining to the betterment of his community and, as a member of the City Council and Chamber of Commerce of Elberton, Ga., contributed largely to the material advancement of the city of his adoption. He was an active member of the Methodist Episcopal Church and served on the Board of Stewards for a number of years.

His charming personality won for him scores of friends and admirers in all walks of life. His connection with professional organizations, and with social and fraternal orders, gave him a wide circle of acquaintances who mourn his passing. He was a member of the American Society of Municipal Improvements, the Florida Engineering Society, the Alpha Tau Omega Fraternity, the Rotary Club, the Masonic Order and the Benevolent and Protective Order of Elks.

Mr. Jaudon was deeply interested in his profession and those with whom he was associated. He never failed to reward the meritorious and lend a helping hand to the unfortunate. In his passing, the Society, his family, and his friends sustain an irreparable loss.

In 1900 he was married to Sarah Louise Arnold, of Elberton, in which city he resided until his death.

Mr. Jaudon was elected an Associate Member of the American Society of Civil Engineers on February 4, 1903, and a Member on February 1, 1910.

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* Memoir prepared by the following Committee of the Georgia Section: J. W. Barnett, *Chairman,* and W. A. Hansell, Members, Am. Soc. C. E., and F. H. Frasuer, Assoc. M. Am. Soc. C. E.

ERNEST LESTER JONES, M. Am. Soc. C. E.*

Died April 9, 1929.

Ernest Lester Jones, the son of Charles Hopkins and Ida (Lester) Jones, was born at East Orange, N. J., on April 14, 1876. He received his educational training at the High School in Orange, N. J., and at Newark Academy. Later he matriculated at Princeton University in the Class of 1898, from which institution he received the Bachelor of Arts Degree.

Following the completion of his studies at Princeton University, Mr. Jones was engaged in research, secretarial work, and business for a number of years. Early in 1913 he entered the service of the Federal Government, President Wilson having appointed him Deputy Commissioner in the United States Bureau of Fisheries. He remained in this position until April 14, 1915, when he became Superintendent (title changed to Director in 1919) of the United States Coast and Geodetic Survey, which position he held until his death.

During his residence in Washington, D. C., Colonel Jones served in the District of Columbia Militia, from Private to Major. During a portion of 1918 and 1919 he was on furlough from the Coast and Geodetic Survey, and was commissioned a Lieutenant-Colonel, U. S. Signal Corps. Later, he became Colonel, Division of Military Aeronautics, and served with the American forces in France in the World War. For exceptional services during the war period he was decorated by the King of Italy as Officer of the Order of S. S. Maurizio and Lazzaro, and Fatigue de Guerre (Italy); he was also an Officer of the Legion of Honor (France). Immediately following the cessation of the war, when men's minds everywhere were turned toward matters of rehabilitation, Colonel Jones was among the first to consider the welfare of those who had been at the battle front in Europe, and thus it came about that he was the organizer of the first post of the American Legion (George Washington Post, Washington, D. C.), and also an organizer and incorporator of the National Legion.

As the efficient administrator of a Federal Bureau, Colonel Jones early came to see the need for better conditions and more adequate salaries for employees in the Federal Service, and his advocacy of their cause in bringing these urgent needs to the attention of the proper authorities had an important part in securing remedial legislation. The great improvement in the efficiency of the personnel of the Federal Service, as a result of this legislation, has amply justified the wisdom of his efforts.

It was a part of his philosophy of human affairs that the best work can be done only when men have the best tools and appliances for doing it, and so it was among his basic endeavors while Director of the Coast and Geodetic Survey that the Bureau's engineers be supplied with adequate ships and modern instruments and equipment. These things he achieved in a large measure, the good effects of which are reflected in a larger and better volume of work and a finer spirit of performance by the entire personnel, so that the Bureau now meets the purposes of its being with a growing satisfaction and increased efficiency.

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* Memoir prepared by R. L. Faris, M. Am. Soc. C. E.

Throughout his administration of the Bureau, Colonel Jones exemplified high executive ability. He was outstanding in his loyalty to the work of the organization and to his associates and subordinates, and in turn he engendered in them such sentiments toward himself. He worked constantly for the improvement of the Service under his direction so that the public might thereby be better served. He was positive in responding personally to whatever seemed necessary to advance each class of work and always co-operated with those of his associates who were making progress. He spared himself no amount of effort and toil to attain the things he thought were right and needful to be done. He had a humane and sincere sympathy for all who requested his assistance. Although firm in his opinions, yet he was considerate of the views of others. He was loyal alike to those whom he served and to those who served under him, and also to his own promises and obligations in that he gave the best that was in him in all his endeavors.

In addition to his duties as Director of the Coast and Geodetic Survey, Colonel Jones was also Commissioner of the International Boundary between the United States and Canada and Alaska and Canada, from February, 1921, until his death. He was a member of the Aerial Patrol Commission of the United States, and a member of a number of Government and scientific missions, one of the last of which was as a delegate to the International Geographic Congress, at Cambridge, England, in 1928.

Colonel Jones was a member of a large number of organizations and societies, which included scientific, engineering, social, patriotic, and outdoor recreational purposes, showing thereby a wide range of active human interest and usefulness. Among the organizations of which he was an active member may be mentioned the Washington Academy of Sciences; the Philosophical Society of Washington; the American Association for the Advancement of Science; the Washington Society of Engineers; the National Geographic Society; the Meteorological Society; the American Fisheries Society; the National Association of Audubon Societies; the Society of American Military Engineers; the Military Order of the World War; and the Society of Mayflower Descendants. His membership in various clubs and civic organizations, also eloquent evidence of his wide interest in his fellow man, included the National Press Club, the Explorers Club (New York), the Aero Club of America, the Cosmos Club, and the Federal Club.

He was the author of the following Government publications: "Alaska Investigations"; "Hypsometry"; "Elements of Chart Making"; "Safeguard the Gateways of Alaska"; Earthquake Investigations in the United States"; and "The Neglected Waters of the Pacific". In addition, he also was the author of the following (unofficial) papers: "Evolution of the National Chart"; "Science and the Earthquake Perils"; and "Aerial Surveying".

In 1919, he was granted the honorary degree of Master of Arts from Princeton University with the following citation:

"Ernest Lester Jones, Director, United States Coast and Geodetic Survey, the oldest scientific agency of our Government, writer on our coastal waterways bordering the Pacific Ocean, a resourceful administrator, increasing largely our supply of reliable maps and supervising the use of new devices for making our waters safer, notably by detecting the perilous submerged pinnacle rocks; a Colonel in the Army during the war, on active service in France and Italy, decorated by the King of Italy, awarded the Diploma of Merit by the Aerial League of America, recommended for the French Croix de Guerre; most recently instrumental in helping to form the American Legion to perpetuate American Liberty".

As a token of its personal regard for Colonel Jones, the following resolutions were adopted by the personnel of the United States Coast and Geodetic Survey and the International Boundary Commission:

Whereas, on April 9, 1929, the death of Colonel Ernest Lester Jones, Director of the United States Coast and Geodetic Survey and Commissioner of the International Boundary Commission, United States-Alaska and Canada, has deprived the nation of an earnest, fearless, and efficient public servant, who always placed devotion to duty above considerations of personal welfare; and,

Whereas, under Colonel Jones' inspiring leadership these organizations have made outstanding contributions both to the immediate public welfare and security and to the better knowledge of scientific truth on which the future of human progress in large measure depends; and,

Whereas, his deep human understanding and sympathy endeared him to all with whom he came in contact, so that those who knew him best loved him most;

Therefore, Be It Resolved, That we, for and in behalf of his friends and associates in the Coast and Geodetic Survey and the International Boundary Commission, unite in expressing our profound conviction that in the passing of Colonel Jones the Administrative Government of the United States has suffered the loss of an executive of rare ability and achievement, and that we have been deprived of a true friend and counsellor;

And Be It Further Resolved, That we extend to his bereaved family our deepest sympathy and express to them our profound sorrow and our heartfelt understanding of the great loss they have suffered.

On September 28, 1897, he was married to Virginia Brent Fox, of Louisville, Ky. He is survived by his widow and two daughters.

Colonel Jones was elected an Associate Member of the American Society of Civil Engineers on April 3, 1922, and a Member on October 12, 1925.

WALSTAN EMILE KNOBLOCH, M. Am. Soc. C. E.*

DIED JUNE 3, 1929.

Walstan Emile Knobloch was born on St. Bridgette Plantation, the family home, in Terrebonne Parish, Louisiana, on December 10, 1862, the only son of Judge Arthur F. and Amelia (Thibodaux) Knobloch. His maternal grandfather, Henry Schyler Thibodaux, was the founder of the Town of Thibodaux, La., and, at one time, served as Governor of Louisiana.

In Mr. Knobloch's early childhood, the family moved to Thibodaux where, as a boy, he attended Thibodaux College, later entering Tulane University, at New Orleans, La., from which he was graduated in 1883. At the time of his graduation he received a gold medal for his proficiency in mathematics.

In order to gain experience, Mr. Knobloch served with the Louisiana State Board of Engineers, without remuneration, from October, 1884, to February, 1886. Subsequent to this time he was engaged as follows: From February to October, 1886, as Computer in the United States Surveyor General's Office, and from October, 1886, to March, 1887, as Recorder in the United States Engineer Office at New Orleans; from March to May, 1887, as Leveler for the Texas and Pacific Railroad Company; from May to September, 1887, as Transitman with the U. S. Engineers, at New Orleans; from November, 1888, until May, 1889, as Resident Engineer on the Costa Rica Railway, having had charge from December, 1888, to May, 1889, of the construction of the Eastern Division of this road, which included the design of all iron bridges on the line; from May, 1889, to August, 1892, as Resident Engineer on the New Orleans and Northwestern Railway, Assistant Engineer on the construction of the Natchez Sewer, Supervisor of Tracks on the Louisville and Nashville Railroad, Transitman on the survey of the Sabine and Marshall Railroad, and, also, Transitman on the survey of the Georgia, Tennessee, and Illinois Railway.

In August, 1892, Mr. Knobloch again entered the employ of the Federal Government in the U. S. Engineer Office, at New Orleans, as Surveyor in the Fourth Mississippi River District, his duties consisting of staking out and supervising the construction of levees from Red River Landing to New Orleans. In May, 1900, he was promoted to the position of Junior Engineer Superintendent in responsible charge of all levee work and of hydrographic surveys in the same District. In July, 1902, Mr. Knobloch was given additional charge of the improvement of the Amite, Bayou Manchac, Natabany, Tickfou, and the Tchefunct Rivers, and of the survey and construction of that part of the Intercoastal Canal, which is in Louisiana.

Continuing his work with the U. S. Engineer Office, he was advanced to the position of Superintendent. In 1923, while thus engaged, Mr. Knobloch suffered a stroke of paralysis and, on account of continued ill health, was forced to resign in 1926. He moved to his home in Thibodaux, where he resided with a sister, Fanny Knobloch, until his death on June 3, 1929. Mr. Knobloch never married.

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* Memoir prepared from information on file at the Headquarters of the Society.

In connection with his work with the Mississippi River Commission, Mr. Knobloch was considered to be one of the most competent and best-informed engineers in that service. He was an expert at levee construction and one of the first advocates of building levees by machinery. It was through his efforts that the use of such construction was extended through the Mississippi Valley below Vicksburg, Miss. His ability in forecasting river stages was almost uncanny, and in critical emergencies during flood periods his judgment many times saved a situation that seemed hopeless.

As a man, Mr. Knobloch was exceedingly modest and like many others his ability and achievements were little known beyond his own circle of intimates. His loss is deeply deplored by his fellow engineers and by others who knew him as a faithful and conscientious worker and a sincere friend.

Mr. Knobloch was elected a Member of the American Society of Civil Engineers on October 2, 1907.

JOHN WILLIAM LIEB, M. Am. Soc. C. E.*

Died November 1, 1929.

John William Lieb was born in Newark, N. J., on February 12, 1860, the son of John William and Christine Zens Lieb. He was educated in the local public schools and the Newark Academy, following which he entered the Stevens High School at Hoboken, N. J., graduating in 1876. He then entered the Stevens Institute of Technology, from which he was graduated with the Class of 1880, receiving the degree of Mechanical Engineer.

During his college course Mr. Lieb became interested in the new discoveries which at that time were being made public in the electrical field and shortly after his graduation he entered the employ of the Brush Electric Company in Cleveland, Ohio, where he worked on the arc-light systems as a Draftsman. He soon saw that these systems had very nearly reached the limit of their development and after a few months, in January, 1881, he entered the employ of Thomas A. Edison, of the new Edison Electric Light Company, as Draftsman in the Engineering Department, at 65 Fifth Avenue, New York, N. Y. Here, he assisted in making the plans for the "Jumbo" dynamos and also in the design of the switches, regulating apparatus, and other electrical equipment for the new Pearl Street Station in New York which was to be the first central station in the world for the general distribution of electrical energy for utilization as light, heat, and power from an underground system of mains. Later in the year, he was transferred to Mr. Edison's Experimental and Testing Department at the Edison Machine Works, in Goerck Street, New York, and was engaged in researches and investigations on electric meters, regulators, and electrical apparatus, under the personal direction of Mr. Edison. Early in 1882, Mr. Lieb was assigned to the construction of the Pearl Street Station, in general charge of the planning and installation of the electrical equipment. When this Station was put into regular operation on September 4, 1882, under the auspices of the Edison Electric Illuminating Company of New York, he was given the appointment of Electrician in charge of its operation.

In November of the same year he was selected by Mr. Edison to proceed to Milan, Italy, to direct the design and installation of the first Edison Station in Milan; and on the organization of the Italian Edison Company (Societa Generale Italiana di Electricita, Sistema Edison), he was appointed its Chief Electrician, then Chief Engineer, and, finally, Technical Manager and Director of Stations. The Milan Station, which was put into regular service in March, 1883, was for a considerable time the largest and most successful electric light and power station in Europe. Mr. Lieb participated in the tests of the Goulard and Gibbs transformers at the Turin Exposition in 1884, and the Milan Company under his direction was among the first to install, in 1886-88, an alternating-current system of distribution, utilizing, among the very first applications, the connection of transformers in parallel according to the Zipernowsky, Deri, and Blathy patents, exploited by Ganz and Company, of Budapest, Hungary. He also did pioneer work in the transmission of high-tension alternating current at 2 400 volts, through concentric underground cables, to consumers located several miles from the central station. Under Mr. Lieb's direction there were also undertaken in Milan some of the first experiments in operating large direct-driven alternators in parallel.

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* Memoir prepared by Geo. A. Orrok, M. Am. Soc. C. E.

The Thomson-Houston Series Arc System, with arc-light machines supplying 50 lamps in series, received here one of the earliest applications in Europe, and later was extended throughout the whole city from a separate arc-light station. The earlier arc-light poles had always been offensive to his æsthetic sense and Mr. Lieb inaugurated the use of artistically designed poles which were much more pleasing to the eye. In 1893, there was put into regular service in Milan under Mr. Lieb's direction, one of the earliest trolley car systems in Italy, furnishing regular service from the Piazza del Duomo to outlying districts. The Italian Edison Company, which was at first only an enterprise installing arc and incandescent lighting plants in cities and industrial establishments all over Italy, later acquired its own manufacturing works for the production of dynamos, motors, and electrical apparatus of all kinds, including the manufacture of incandescent lamps. Mr. Lieb's interest in the incandescent lamp, which had manifested itself in his college days and continued during his work with Mr. Edison, was now used to good purpose, and his executive genius and organizing ability ensured the success of these establishments.

In the meanwhile he had returned to New York to be married to Minnie F. Engler, on July 29, 1886, taking her back with him to Milan where he continued his work in popularizing and extending the use of electric light, heat, and power. It was during this visit to America that Mr. Lieb became a member of the American Society of Mechanical Engineers. He was elected a member of the American Institute of Electrical Engineers the next year. In his connection with the Italian Edison Company, he served as adviser for many financial and industrial corporations engaged in the electric business where his active and enthusiastic mind united with the judgment of manager and executive, assured sound methods in the exploiting and extension of the electric industry. Already the possessor of three languages, German, French and English from early life, he now had become master of Italian, even speaking the local dialects with considerable ease.

In 1894, Mr. Lieb returned to America as Assistant to Mr. R. R. Bowker, then Vice-President and Executive of the Edison Electric Illuminating Company of New York. The next four years were most fruitful in the development of the use of electric light and power in New York City and Mr. Lieb made three visits to Europe with other officials of the company. This was preliminary to the design and construction of the Waterside Station and the adoption of high-tension alternating-current generation, with rotary converters and low-tension direct-current sub-stations for the Edison System. These years also saw the first storage battery installed on that System.

On January 1, 1898, the control of the Company passed into the hands of Mr. A. N. Brady, with Mr. T. E. Murray as Vice-President. At this time, Mr. Lieb became Third Vice-President and General Manager, serving in this capacity until the Company's re-organization as the New York Edison Company in 1901. In the re-organized Company he became Associate General Manager, Vice-President, and General Manager, and, finally, Senior Vice-President, which office he held at the time of his demise. During this period he had general charge of the installation and operation of the power plants and distributing systems in New York and was the executive in charge of the Company's technical operations, with supervision over all technical research and developmental work.

In addition to being Senior Vice-President and a Director of the New York Edison Company, Dr. Lieb was President and Chairman of the Board of The Electrical Testing Laboratories; Vice-President and member of the Board of Directors of the Yonkers Electric Light and Power Company; member of the Board of Directors of the United Electric Light and Power Company, and of the New York and Queens Electric Light and Power Company; Vice-President of the Edison Electric Light and Power Installation Company; Director and member of the Executive Committee of the International Power Securities Corporation; Director of the Brush Electric Illuminating Company, the Empire City Subway Company, and the Consolidated Telegraph and Electrical Subway Company.

He was a Past-President and Fellow of the American Institute of Electrical Engineers; Past-President of the Association of Edison Illuminating Companies, National Electric Light Association, the New York Electrical Society, and the Edison Pioneers; Past Vice-President of the American Society of Mechanical Engineers and Fellow of the New York Academy of Sciences; Trustee of the Stevens Institute of Technology; Trustee and Vice-President of The Museums of the Peaceful Arts; and Trustee of the Italy-American Society. He was a member of the Illuminating Engineering Society; the Franklin Institute of Philadelphia; the American Association for the Advancement of Science; and numerous other professional and civic organizations, National and local. He was an Honorary Member of the Society of Italian Engineers and Architects, and of the Society of Italian Railway Engineers; Vice-President, Union Internationale des Producteurs et Distributeurs d'Energie Electrique; a member of the Elektro-technischer Verein, Associazione Electrotecnica Italiana, the Institution of Electrical Engineers of Great Britain, the Royal Society of Arts, and the Newcomen Society of London.

Dr. Lieb was for many years past the Chairman of the Lamp Committee of the Association of Edison Illuminating Companies, and during the World War was Chairman of the National Committee on Gas and Electric Service, representing in Washington, D. C., the public utility companies (gas, electric, and street railway service) of the United States in their co-operative relations with the various departments of the Government. He was a member of the Committee on Public Utilities of the U. S. Chamber of Commerce, on which he represented the National Electric Light Association as its National Councillor; Chairman of the Association's delegation to the International Chamber of Commerce; representative of the Association on the International Association of Producers and Distributors of Electrical Energy; member of the

New York Chamber of Commerce and of its Committee on Internal Trade and Improvements; member of The Merchants Association of New York and Chairman of its Industrial Committee; member of the National Industrial Conference Board and Chairman of the Joint Fuel Committee, representing the National Public Utility Associations; member of the Committee to Study Plans for Co-operation between the War Department and the National Association of Manufacturers; and a member of the Committee on Standardization Survey, the American Society of Military Engineers, and the National Research Council.

He also served in international matters as follows: Chairman, Section "E", International Electrical Congress, St. Louis, Mo., 1904; Representative, U. S. Department of State at the International Railroad Congress, Rome, Italy, 1922; Member, U. S. Executive Committee, World Power Conference, London, England, 1924; U. S. Delegate, International Congress on Illumination, Geneva, Switzerland, 1924; Chairman, Reception Committee, International Electrotechnical Commission Meeting, New York, N. Y., 1926; Vice-President, International Association of Producers and Distributors of Electrical Energy, Paris, France, 1928; and Executive Chairman, American Organization Committee, World Engineering Congress, Tokyo, Japan, 1929.

For his pioneer work in connection with the introduction of electric light and power service throughout Italy and the installation of the electric trolley system in Milan, Dr. Lieb was decorated by the Italian Government, being made Knight Commander of the Royal Order of the Crown of Italy and, later, being promoted to Grand Officer. In 1925 he was made "Ufficiale" of the Order of S.S. Maurizio e Lazzaro, and, in 1928, "in recognition of the services he has rendered in furthering professional industrial co-operation between French and American engineers and in facilitating the interchange between them of knowledge and experience regarding the construction and operation of central-station systems for the generation and distribution of electrical energy for light, heat, and power," was honored by the French Government, receiving the decoration of Officer of the Legion of Honor.

Dr. Lieb was most sympathetic with the youthful aspirant to a position in the Engineering Profession. He valued highly the training he had received at the Stevens Institute of Technology and maintained a very active interest in engineering education and the progress of his Alma Mater. He served as President of the Alumni Association in 1896-97 and as Alumni Representative on the Board of Trustees from 1908 to 1911. In 1921, he received, from his Alma Mater, the honorary degree of Doctor of Engineering, and, in 1927, became a life member of the Board of Trustees.

He was equally interested in technical Society affairs, becoming, as stated, a member of the American Society of Mechanical Engineers in 1886, and of the American Institute of Electrical Engineers in 1887. In the American Institute of Electrical Engineers he was elected Manager for the terms 1896-1899 and 1901-1903; Vice-President from 1899-1901 and from 1903-1904; and President in 1904-1905. In 1913, he became a Fellow of the Institute. He was elected Manager of the American Society of Mechanical Engineers for the term 1903-1906, and Vice-President for the term 1906-1908. He was greatly interested in the affairs of United Engineering Society and particularly of its Library, serving continuously on this Committee from 1909 to 1917. Outside these activities, he gave freely of his time and advice in the furtherance of the work of these technical societies.

Dr. Lieb delivered many lectures on engineering, industrial, and economic subjects in the leading universities and technical schools in the United States, and contributed many reports, papers, and discussions to the publications of professional and learned societies. On February 4, 1924, he was awarded the Edison Medal by the American Institute of Electrical Engineers for his work in connection with the development and operation of electric central stations for illumination and power.

As a manager and executive, Dr. Lieb held very definite opinions regarding the position of utilities or public service corporations as trustees for both the public and the investors. Coming into the business during its inception, he knew the experimental and speculative side and the long, arduous years before an adequate return became possible on the funds invested. As a public service corporation is practically without competition, he held that service must be maintained as perfect as possible and at as low a rate as was consistent with fair returns on the investment. He early saw the futility and fallacy of municipal ownership blended with political control, and dependent on the tax fund to finance deficiencies. He saw his company paying a sufficient dividend to ensure the securing of additional capital when required and utilizing the best methods and devices for efficient and continuous service to the public at the lowest price conformable with a sound corporate policy.

As General Manager, the selection and administration of the Edison Company's personnel became his personal responsibility, which he considered as important as his responsibility to his stockholders and to the public. His wide sympathies were well known and his office was open to everybody, even to the youngest office boy, who had a grievance. Having worked as a mechanic, he knew the workman's point of view and respected it. His dealings with organized labor were sympathetic, but a suspicion of the "ca'canny" roused him to antagonism. He who gave of himself without limit and of the best that was in him, could not lightly suffer the cheat or "clock watcher".

With the vision of the pioneer, the well considered knowledge and training of the scientist, and the wide sympathies of the humanitarian, Dr. Lieb endeared himself to every one with whom he came in contact. In a conference or argument, he had the faculty of picking out, as it were, the underlying idea, the main trend, the crux of a situation, and divesting it of all extraneous matter, going straight to the root of things. His reasoning was never oblique, but logical and cogent, sparing none, not even himself, in the strength of his analysis of a situation.

His sympathies were wide, ranging over the entire realm of human activities; his versatility and general knowledge might be termed encyclopædic. His English vocabulary was perhaps his most wonderful achievement, and his writings have always been models of clearness and good expression, in his use of the right word in the right place to obtain just the shade of meaning he desired.

While Dr. Lieb's first interest was always the electric industry, and particularly the Edison System, he had many hobbies, among which history, economics, art and archæology were perhaps the most important. His long residence in Italy, with its archæological monuments, and many survivals of mediæval times and the Renaissance, had given him a wide knowledge of these great periods of the world's history. He early became acquainted with the work of Leonardo da Vinci and studied his notebooks, learning the old Italian dialect in which they were written and becoming one of the foremost authorities on the work of that universal genius. He became greatly interested in the Etruscan monuments and the earlier Hallstatt remains. His vacations were spent in research on art and history, visiting most of the sites of ancient cities and getting first-hand knowledge of the progress of civilization. He early joined the Raccolta Vinciana of Milan and maintained a most active interest in this Society. He was also greatly interested in the work of Perret, the volcanologist, and in the development at Llarderello, Italy, utilizing the interior heat of the earth for the production of power.

Dr. Lieb's reading was most extensive because he early acquired a capacity for rapid reading, which is decidedly uncommon at the present time. When this was added to a very retentive memory and a most wide-awake and far-reaching interest in both the modern and ancient world, the scope of his activities may be fully appreciated.

Mr. Edison, paying tribute to him, says:

> "John W. Lieb was one of my most able assistants, and in the early days I used him in the development of the Pearl Street Central Station. He was a most energetic and active man and had such a comprehensive understanding of my electric lighting system that I selected him to go over to Italy and establish the Milan Central Station. He made a great success of the installation and of the introduction of the system into Italy. One of his notable achievements was the lighting of La Scala, the great opera house of Milan.
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> "His work in Italy was of a pioneering nature and led to the promotion of the great hydro-electric system which has been so extensively exploited in that country. His work there was but the beginning of a great and noteworthy career which, unfortunately, has come to an untimely end.
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> "He was a man of fine and sterling character, and I always had a very warm regard for him."

Dr. Lieb is survived by his widow; two daughters, Minnie E. and Julia C. Lieb; a son, Adolph W. Lieb, who was married in 1928 to Eleanor Wilcox; and a granddaughter. A brother, Oscar J. Lieb, and two sisters, Mrs. Charles Oscar Baldwin and Miss Anna Lieb, also survive him.

Dr. Lieb was elected a Member of the American Society of Civil Engineers on October 5, 1898.

ALBERT LUCIUS, M. Am. Soc. C. E.*

Died February 2, 1929.

Albert Lucius, the son of Frederick and Amalia (Fudikar) Lucius, was born on October 9, 1844, at Erfurt, Prussia. He was graduated from the Frankfort Polytechnic, Germany, in 1863, and was employed as a Mechanical Draftsman until 1865, in which year he came to the United States and made his home in New York, N. Y.

Mr. Lucius was engaged as a Mechanical Draftsman with the Novelty Iron Works, New York, from 1865 to 1870, at which time he severed his connection with that Company to accept the position of Superintendent of the Petersburg Iron Works, at Petersburg, Va. He remained with the Petersburg Iron Works until 1875 and then returned to New York City to accept the position of Assistant Engineer with the New York Elevated Railway. He held this position until 1877, at which time he was promoted to be Principal Assistant Engineer of the same road, where he remained until 1880.

During 1880 and 1881, he was Engineer of Construction for the Brooklyn Elevated Railway. From 1881 until 1883 he served as Principal Assistant Engineer of Bridges for the New York, West Shore, and Buffalo Railway Company, under the late Walter Katté, M. Am. Soc. C. E., Chief Engineer.

Among the personal effects of Mr. Lucius a letter addressed to "Whom It May Concern", written by Mr. Katté, and endorsed by the late Cyrus W. Field, was found, from which the following is quoted:

> "I take pleasure in certifying that Mr. Albert Lucius, Civil and Mechl. Engineer, is eminently qualified to satisfactorily and skillfully perform all the duties of a Chief Engineer of Railways, Bridge Structures, Elevated Railway, and their appertaining machinery. I tendered him the position of Chief Asst. in the Dept. of Iron Bridging Structures generally, in the exercise of these duties he was responsible for correct designing and execution of details of all the Iron Structures on those lines, many of which were of great magnitude, and calling for engineering ability of a high order. It gives me pleasure to state that his performance of them was in every way commendable and satisfactory, and if I were asked to nominate a Chief Engineer of an Elevated R. R. requiring the services of a thoroughly qualified Engineer he is exactly the man I should first name."

It is of interest to find two such notable engineers combining to endorse the technical ability and qualifications of a fellow member of that profession.

During the latter part of 1883, Mr. Lucius was engaged by the late James B. Eads, F. Am. Soc. C. E., President of The Eads Concession Company, to accompany him to London, England, to assist in the plans, specifications, and negotiations for the Tehuantepec Ship Railway. This railway was to be located across the Isthmus of Tehuantepec, Mexico, for the purpose of transporting vessels overland between the two oceans. The ship railway project, however, did not materialize, and Mr. Lucius returned to New York City the next year and became a member of a firm of bridge contractors. He remained with this firm until 1886, at which time he opened an office in New York

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* Memoir prepared by A. R. Raymer, M. Am. Soc. C. E.

City as Consulting Engineer and continued in active practice in that city until his death.

Although Mr. Lucius devoted himself in his private practice to all forms of structural steel designing, he may be said to have specialized in the design of steam railway bridges, inasmuch as he designed such structures for the following: The Bessemer and Lake Erie Railroad Company; the Boston and Albany Railroad Company; the Franklin and Clearfield Railroad Company; the Lake Erie and Eastern Railroad Company; the Lake Erie and Pittsburgh Railroad Company; the Lake Shore and Michigan Southern Railway Company; the Maine Central Railway Company; the New York Central Railroad Company; the New York, Ontario, and Western Railway Company; and the Pittsburgh and Lake Erie Railroad Company. Perhaps his most notable work was his design of the double-track cantilever bridge over the Ohio River at Beaver, Pa., for the Pittsburgh and Lake Erie Railroad Company.*

Mr. Lucius was inherently an engineer, a clear and an original thinker, and, as a man, most excessively modest. It may be truly said of him that he made many real and genuine contributions to the Engineering Profession during his long and notable career.

On August 7, 1873, Mr. Lucius was married to Octavia Wallace Sturdivant, a resident of Petersburg, Va., who died in New York during 1919. He is survived by his five children, Frederick J., William W., and Edward C. Lucius, Mrs. William E. Goeringer, and Mrs. William Grimm.

Mr. Lucius was elected a Member of the American Society of Civil Engineers on March 3, 1886.

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* *Transactions*, Am. Soc. C. E., Vol. LXXIII (1911), p. 136.

WILLIAM HENRY LUSTER, Jr., M. Am. Soc. C. E.*

Died December 12, 1928.

William Henry Luster, Jr., the son of William H. and Elizabeth H. (Yates) Luster, was born at Elizabeth, N. J., on October 11, 1863. He was descended on his father's side from old Colonial stock.

Mr. Luster's early school days were spent in Elizabeth, and his later education obtained at Rutgers College, New Brunswick, N. J., from which he was graduated in 1884 with the degree of Bachelor of Science. In 1890, he received the degrees of Master of Science and Civil Engineer from his Alma Mater.

Following his graduation from college, Mr. Luster was engaged from December, 1884, to December, 1887, as Topographer with the United States Geological Survey and from December, 1887, until 1890, as Assistant to the Division Engineer of the Morris and Essex Division of the Delaware, Lackawanna, and Western Railroad Company, in charge of the maintenance of way and construction of 10 miles of new road. In January, 1890, he entered private practice and was engaged on the construction of county roads in Union County, New Jersey, and in the vicinity of New York City. He had practically full charge of four miles of construction work and laid out and graded Colonial Park, N. J., for Mr. J. R. McPherson. During this period, he was also engaged on sewer construction and had charge of the building of coal pockets for Mr. L. M. Palmer, of Brooklyn, N. Y., as well as foundations of the power house for the Edison Electric Illuminating Company, at Bay Ridge, N. Y.

In June, 1896, he was appointed a Topographer with the New York State Land Survey, which position he retained until June, 1899, when he was elected City Engineer for Elizabeth, N. J. In this capacity he had complete charge of the design and construction of public works the value of which was more than $500 000. From January, 1913, to November, 1926, he was engaged in private practice as Engineer for the Towns of Roselle Park, Roselle, and Hillside, N. J. He made the plans for the sanitary sewer for the latter town, which finally was completed successfully by him in November, 1928.

Mr. Luster was a member of the Chi Phi Fraternity, the American Road Builders' Association, and the American Society of Municipal Improvements. He was also a Director of the Citizens' Building and Loan Association of Elizabeth and a senior partner of the firm of Luster and Luster. Aside from these affiliations, Mr. Luster was interested in hunting, photography, traveling and fishing, and, as a hobby, took great delight in the making of fine fishing rods. He was a Democrat in politics, a Rotarian, and a member of the Presbyterian Church.

In December, 1889, Mr. Luster was married to Bertha Overton Looker, of Elizabeth, who, with two sons, Clifton H. and Eric W., survives him.

Mr. Luster was elected an Associate Member of the American Society of Civil Engineers on October 3, 1900, and a Member on April 2, 1902.

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* Memoir prepared from information on file at the Headquarters of the Society.

ALEXANDER MacDONALD, M. Am. Soc. C. E.*

DIED MAY 11, 1929.

Alexander MacDonald, the son of John and Anne (Fraser) MacDonald, was born in Inverness, Scotland, on March 17, 1877. He was one of seven children. He received his early education in the public schools of his native city and was graduated in 1895 from the Nairn Academy.

Mr. MacDonald came to America in 1900, and started to work in New York, N. Y., as a street-car conductor, studying engineering through a correspondence school in the evenings. In 1902, in response to an advertisement, he entered the employ of E. W. Hess, M. Am. Soc. C. E., of Clearfield, Pa., as an Assistant Engineer. He remained with Mr. Hess until September, 1904, when he entered the Rennselaer Polytechnic Institute, at Troy, N. Y., to take a special engineering course. At Rensselaer, he achieved the honor of being the first President of the Class of 1908; he was also a member of the Rensselaer Society of Engineers.

In 1905, Mr. MacDonald left Rensselaer Polytechnic Institute, going back to the employ of Mr. Hess, at Clearfield, and until 1909, he had charge of a Field Engineering Corps on railroad location and construction, the installation of water lines and water supply reservoir, and the development of coal and clay mines and of maps pertaining thereto.

Mr. MacDonald was engaged by the North Western Mining and Exchange Company, of Du Bois, Pa., in 1909, and, until 1911, had charge of the making of a complete survey of 3 500 acres of coal land, diamond drilling, and location, the making of plans of a new shaft, with the necessary buildings and equipment using electric power. He received a promotion in 1911, being made Chief Engineer of the North Western Mining and Exchange Company and of the Blossburg Coal Company, at Du Bois, subsidiaries of the Erie Railroad Company, with holdings of approximately 90 000 acres of coal, oil, and gas lands. His duties included the supervision of mine projections and development, the design and construction of a steam turbine-driven central power plant, transmission lines and sub-stations, and all other equipment incidental to an electric mining operation. He remained in this position until 1917, when he became Chief Engineer for the Pine Run Coal Company, at New Bethlehem, Pa., where for a year he had charge of the location and drilling of gas and oil lands, in addition to the development of the mines of this Company.

In 1918, Mr. MacDonald became Chief Engineer of the Keystone Coal and Coke Company, of Greensburg, Pa., which at that time owned twenty plants. In 1922, this Company purchased the coal holdings of the Jamison Coal and Coke Company in the Greensburg Field and the addition of this property, which adjoined the original holdings, presented an interesting problem in economy of power, by centralizing the power consumed at nine of the plants located within a reasonable distance of each other. Mr. MacDonald also recommended the purchase of power from the West Penn Power Company and built additional transmission lines and installed sub-stations, transformers, and metering equipment to consolidate the purchased power at a single metering point.

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* Memoir prepared by Allen S. Davison, Assoc. M. Am. Soc. C. E.

Although Mr. MacDonald's connection with the Keystone Coal and Coke Company continued until his death, on several occasions he had been called upon to act as Consulting Engineer on problems in Canada as well as in other parts of the United States.

Mr. MacDonald was considered one of the most capable mining engineers of Western Pennsylvania. He was a member of the American Institute of Mining and Metallurgical Engineers. He was also a member of the Hannastown, Greensburg, and Oakmont Country Clubs. He attended the Presbyterian Church.

On October 1, 1927, he was married to Florence Helen McDonald, the daughter of Mr. and Mrs. Frank A. McDonald, of Carnegie, Pa., who survives him. He is also survived by his parents, four sisters, and two brothers.

Mr. MacDonald was elected a Member of the American Society of Civil Engineers on January 17, 1927.

DAVID EDWARD McCOMB, M. Am. Soc. C. E.*

Died January 27, 1929.

David Edward McComb, the son of James E. and Martha (Jorden) McComb, was born on August 23, 1850, at Chelsea, Mass. On one side of the family he was of Scotch ancestry, his father having been born in Scotland; on his mother's side he was of French lineage. His father, a school teacher, died in 1852 and, in 1855, the family moved to Washington, D. C., which thereafter was Mr. McComb's home.

He attended private and public preparatory schools in Washington until he was fifteen years of age. In 1865, he entered the Navy Yard at Washington, as a Minor in Steam Engineering, an educational course provided by the Navy Department at that time for the training of candidates for commissions as Steam Engineers. After three years of this class work, however, Congress provided, in 1868, that future appointments to such commissions should be made only from graduates of the Naval Academy at Annapolis, Md.

Mr. McComb's training, thus interrupted in 1868, had consisted largely in experimental work under William Shock, Chief Engineer, U. S. Navy, on strains in Bessemer steel, and under S. M. Green, U. S. Navy, on calorific conductibility of boiler-plate materials. He also completed a two years' course in higher mathematics and modern languages at Columbia College, now George Washington University, in Washington. From 1868 to 1870, he was engaged under John A. Coli, Civil Engineer, in Washington, on general city work; in 1870, as Assistant to John A. Partridge, on the construction of the Baltimore and Potomac Railroad; in 1871, as Assistant to the late Thomas Doane, M. Am. Soc. C. E., on the construction of the Burlington and Missouri River Railroad, in Nebraska; in 1872, as Topographer on the Jersey Shore, Pine Creek, and Buffalo Railroad, and as Engineer in Charge of the Fredericksburg, Orange and Charlottesville Railroad; and in 1873-74 as Resident Engineer on the Washington City and Point Lookout Railroad.

He entered the municipal engineering service of the District of Columbia on September 1, 1874, and continued therein except for a four-year interval, until his retirement on reaching the age limit on August 20, 1928—a half century of professional service to this municipality. The first thirty-four years of this service, from 1874 to 1908, was in the Sewer Division, of which he became Superintendent in 1888. As such, he devised a formula, based on the experimental work of Burkli-Ziegler, modified to suit local conditions, which is still applied as well as the Kutter formula, to drainage design in the Department.

Following extensive inspections of the existing sewage disposal installations in the United States, England, and France, Mr. McComb recommended a disposal project for Washington in 1890, the engineering elements of which were followed, with but little material variation, by the report made later in that year by a board of sanitary engineers appointed from the country at large by the President to consider this subject. The trunk drainage and sewer mains, subaqueous crossing of the Anacostia River, the subaqueous outlet system and sewage pumping station constituting this disposal project were located, designed, and constructed under his supervision.

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* Memoir prepared by C. B. Hunt, M. Am. Soc. C. E.

On May 15, 1908, Mr. McComb was appointed Chief Engineer of Sewer and Paving Work in Havana, Cuba, where he served as such for four years, the appointment having been made during the period of the second American intervention. During his incumbency in this position he practically completed the subaqueous sewer crossing of Havana Harbor, the sewage pumping station and the tunnel under Cabanas Ridge, as well as many of the urban sewers and also the beginning of the paving of the city street system. His selection for this duty was made by Colonel William M. Black, Corps of Engineers, U. S. A., M. Am. Soc. C. E. (now Brigadier-General, U. S. A. (*Retired*)), Adviser of the Department of Public Works of Cuba, based on his knowledge of Mr. McComb's high professional ability and uncompromising integrity of character.

On July 1, 1912, Mr. McComb returned to the municipal service of the District of Columbia as an expert on asphalt paving work in connection with the contemplated establishment of a municipal asphalt plant; at the conclusion of this duty, on January 1, 1913, he was appointed Engineer of Bridges, a position that he held with distinction for fifteen years until his retirement on account of age. Under his supervision during this period were the structural features of the Dumbarton Bridge, the Meigs Bridge, the Benning Road Viaduct, the South Dakota Avenue and Van Buren Street Bridges over the Baltimore and Ohio Railroad, and many wharves, trestles, and analogous works constructed by the municipality.

Mr. McComb's professional work was marked by sound knowledge of principles, conservative design, conscientious care in execution, and remarkable integrity of administration.

His outstanding personal characteristic was rugged honesty. The circle of his friends was large and he held them throughout his life. He was fond of many sports; playing baseball with notable skill; bicycling throughout Europe and many parts of the United States; and in his later years becoming an enthusiastic golfer. He was affiliated with the Washington Society of Engineers, of which he was a Charter Member, the Capital Bicycle Club, and the Columbia Golf Club.

Mr. McComb was married on August 28, 1877, to Theresa M. Maher who pre-deceased him on March 9, 1924. A daughter, Mary E. (McComb) Fries, survives. He was a devout member of the Roman Catholic Church.

Mr. McComb was elected a Member of the American Society of Civil Engineers on February 7, 1877. His period of membership, extending within a few days of fifty-two years in length, was exceptional and notable; and he left but very few living Society members in date of membership senior to him.

CHARLES NEIL MCDONALD, M. Am. Soc. C. E.*

Died July 5, 1929.

Charles Neil McDonald, the son of Alexander and Johanna McDonald, was born on August 25, 1862, near Sidney, Nova Scotia. As a boy he attended the grade schools of Sidney, and there, too, he served an apprenticeship as a carpenter.

In 1882, after living for a short time in Boston, Mass., Mr. McDonald moved to the Dakotas, where he was employed as Bridge Carpenter and Bridge Foreman by the St. Paul, Minneapolis, and Manitoba Railway Company (now a part of the Great Northern Railway Company).

In 1884, Mr. McDonald moved to the Far West and located at Celilo, Ore., where he entered the employ of the Oregon Railroad and Navigation Company as Bridge Foreman on the construction of stone, concrete, iron, and timber bridges.

In 1890, he was appointed Superintendent of Construction for Robert Wakefield and Company, Contractors, of Portland, Ore., and for eighteen years he continued in active charge of all the numerous large construction projects undertaken by that firm. During this period, Mr. McDonald's work covered a wide scope and included docks and harbor improvements at various points along the Pacific Coast, numerous bridges for the Union Pacific System, all the bridges on the Astoria and Columbia River Railway, and several large structures for the City of Portland. He was also retained from time to time to represent the Chief Engineer of the Oregon Railroad and Navigation Company in the execution of difficult bridge foundations and in the solution of various erection problems.

In 1908, his activities carried him into the Far North. He associated himself with the Copper River and Northwestern Railway Company which was being extended into the interior of Alaska, and was placed in charge of all the bridge construction on that line.

This work, which included the famous Miles Glacier Bridge and the Kuskolina Cantilever, was crowned with notable success in the face of most adverse circumstances. Operations were carried on far from a base of supplies; the season of the year in which climatic conditions were favorable for construction was limited to only a few months; and the work had to go on during the long stormy winters of the Copper River country and in temperatures almost unbelievably low.

Large masses of ice are discharged from Miles and Child's Glaciers into the Copper River during the summer months, and for that reason the Miles Glacier Bridge which crosses that stream was originally designed for cantilever erection. To advance the opening of the railroad a full year, however, the plan was changed, and it was decided to carry on substructure work during the winters and, at the same time, to erect the spans of falsework supported on ice. The plan was a success. The ice moved out in an unusually early spring thaw just twenty minutes after the bridge was self-supporting. This dramatic incident is well described in Rex Beach's novel, "The Iron Trail."

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* Memoir prepared by S. Murray, M. Am. Soc. C. E.

In 1912, Mr. McDonald returned to Portland and again associated himself with Robert Wakefield and Company. For several years he was the active head of that firm in charge of the execution of important river and harbor work and large bridge projects, involving the expenditure of many millions of dollars. Included in this work was the erection of the Harriman Bridge (double-deck railroad and highway lift) across the Willamette River, at Portland, and the Interstate Bridge across the Columbia River, at Vancouver, Wash.

In 1920, he became Vice-President of the Gilpin Construction Company of Portland, and, from that time until his death, he directed its more important operations, among which were bridges across the Columbia, Willamette, and other large rivers of the Pacific Northwest.

Mr. McDonald possessed to a rare degree the good judgment that is acquired only in the stern school of experience, but he combined with it a technical knowledge and designing ability worthy of a man of great formal training. His counsel was sought by many, especially by the younger men, whom he seemed to understand. His life was marked by a succession of great achievements, bold and daring in conception, taxing amazing resources of skill and energy in execution, yet uniformly successful in realization.

Deeply religious, and of a modest and retiring disposition, generous, honorable, and upright in all things, he was an inspiration to those who came in contact with him. No man was loved more by his associates. His passing has left an empty place which they feel can never quite be filled.

In 1890, he was married to Carrie Harrison, who survives him, together with two daughters, Mrs. Joan Atwater, of Eugene, Ore., and Mrs. Jessie Acklen, of Raymond, Wash.

He was a member of various Masonic bodies and other fraternal orders, and was affiliated with the Methodist Episcopal Church.

Mr. McDonald was elected a Member of the American Society of Civil Engineers on January 17, 1927.

LOUIS RANDOLPH McLAIN, M. Am. Soc. C. E.*

DIED JANUARY 9, 1926.

Louis Randolph McLain was born at Washington, D. C., on March 13, 1846, the son of the Rev. William McLain and Mary (Mosby) McLain. His early education was obtained in the schools of Washington, the home of his parents. Later, he entered Hamilton College, Clinton, N. Y., with the Class of 1865, but left at the expiration of his Junior year.

Mr. McLain's first engineering assignment was in December, 1869, when he entered the service of the South Trans-Continental Railroad Company as Draftsman and Office Assistant. In 1870 he was made successively Rodman, Transitman, and Leveler, being engaged on preliminary surveys and location until the consolidation of that road and the Southern Pacific Lines under the name of the Texas and Pacific Railroad Company. Thereafter, until January, 1872, he was Resident Engineer for the Jefferson Division of the latter road, resigning to engage in railroad and other construction work, which continued until 1876. In March of that year he was made Resident Engineer of the New Orleans Pacific Railroad Company, remaining in charge of construction until suspension of the work in June, 1878. He then located and superintended the construction of the Kanomie Narrow Gauge Railroad from Bayou Bouf to Kanomie Landing, on the Red River, Louisiana.

Following its construction, he operated the road until January, 1879, when he resigned to become Assistant United States Engineer of South Pass Jetties, at Port Eads, La. On the completion of this work in August, 1879, he was engaged as Assistant in the improvement of the James River, Virginia. Later, in March, 1880, he was made Resident Engineer for the Becharant and Clifton Forge Division of the Richmond and Allegheny Railroad, now a branch of the Chesapeake and Ohio Railroad.

This work being finished, Mr. McLain became Division Engineer of the Georgia Pacific Railroad Company, now that part of the Southern Railway System connecting Atlanta, Ga., and Birmingham, Ala. Thereafter, his activities were centered in Florida. In 1884, he was made Assistant Chief Engineer of the South Florida Railroad Company, now a part of the Atlantic Coast Line, running from Sanford to Tampa, Fla. When this road was completed in 1885, he was appointed Maintenance of Way Engineer, resigning two years later to enter the field of railroad construction, active at that time in the South.

During the next ten years, Mr. McLain constructed railway lines from Thomasville, Ga., to Montgomery, Ala.; from Thomasville, to Monticello, Fla.; from Savannah to Tybee, Ga.; and in addition a line from Savannah to Columbia, S. C., now a part of the Seaboard Air Line System.

In 1898, he turned his attention to hydraulic mining of pebble phosphate in Polk County, Florida, building, equipping, and operating successfully a plant, conceded by every one having knowledge of the industry, to be the most complete in Florida. Later, in July, 1903, these interests were sold to Savannah capitalists, representing the Peace River Phosphate Company.

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* Memoir prepared by R. F. Smith, Esq., Miami, Fla.

Mr. McLain's retirement from active work followed the sale of his phosphate interests. He then opened an office as Consulting Engineer in St. Augustine, Fla., where he made his home, representing, among others, Belgian capitalists in the direction of their Florida mining enterprises until 1917.

Aside from his professional activities and mining enterprises, Mr. McLain maintained, from 1890 to 1920, a controlling interest in two banking institutions in Florida and Alabama. The success of these, through a long period of years, was largely due to his sound judgment and wise leadership.

Mr. McLain was married on April 29, 1897, to Harriott Rutledge Ravenel, the daughter of Dr. St. Julien Ravenel and Harriott Horry Ravenel, of Charleston, S. C.

He was a member of the St. Augustine Golf Club, the St. Augustine Yacht Club, and the Commonwealth Club of Richmond, Va.

His long, useful life was characterized by thoroughness in the discharge of every undertaking and a sensitive loyalty to his friends and the interests that he served.

Mr. McLain was elected a Member of the American Society of Civil Engineers on February 2, 1881.

GEORGE COTNER MASON, M. Am. Soc. C. E.*

DIED MARCH 10, 1929.

George Cotner Mason, the son of James R. and Mary W. (Cotner) Mason, was born in New York, N. Y., on May 4, 1871. He received his early education in the public schools of New York and was a student in Civil Engineering at New York University from which he received the degree of Bachelor of Science in 1892, Civil Engineer, in 1893, and Master of Science, in 1894.

From 1892 to 1904, Mr. Mason was engaged as Assistant Instructor and Assistant Professor in Civil Engineering at New York University. During this period he laid out its grounds, buildings, and sewer system, and was also employed as Engineer in charge of the erection of the Young Men's Christian Association Naval Branch Building, in Brooklyn, N. Y., the Army Branch Building, at Fortress Monroe, Va., and a residence, stable, and riding ring for Mr. Frank J. Gould, in New York.

In 1904, Mr. Mason moved to Portland, Ore., where he organized the Hurley-Mason Company, serving as its Vice-President and Chief Engineer. The Company was engaged in engineering and contracting work, with offices in Portland and in Seattle, Tacoma, and Spokane, Wash. It completed many important contracts in the Pacific Northwest, notable among which are the Oak Grove Hydro-Electric Plant of the Portland Electric Power Company and Camp Lewis, a 45 000-man Army Cantonment, near Tacoma. The latter was a World War activity and was completed in record time.

The Hurley-Mason Company also constructed many important buildings and structures throughout the Northwest. Among these are the Todd Shipbuilding Plant and the Northern Pacific Railway Station, in Tacoma; the U. S. Veterans' Hospital, at American Lake; the Seattle Construction and Dry Dock Plant, in Seattle; the Sherwood Building and Elks Temple, in Spokane; the Lewiston Hotel, in Lewiston, Idaho; and the Electric, Board of Trade, Gas Company, and Lipman-Wolfe Buildings and the Portland Flouring Mills, in Portland.

In 1926, Mr. Mason retired from the Company and, until his final illness, was engaged in the private practice of his profession, with offices in Portland.

He took a keen interest in civic matters and served as Civil Service Commissioner for the City of Portland from 1916 to 1925. He was also active in the affairs of the Society and was particularly interested in the development of the Student Chapters and in the establishment of closer relations between the profession and the students during their college years. He represented District No. 12 as a Director of the Society from 1923 to 1925.

Mr. Mason was an engineer of independent and courageous judgment, thorough and painstaking in all his work, and these characteristics are evident in the many structures built by his Company.

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* Memoir prepared by Ben S. Morrow, Assoc. M. Am. Soc. C. E.

In 1900, he was married to the daughter of a prominent pioneer family of Portland, M'liss McCracken, who, with one son, Henry M. Mason, Jun. Am. Soc. C. E., survives him.

Mr. Mason was elected a Junior of the American Society of Civil Engineers on March 6, 1894; an Associate Member on May 3, 1899; and a Member on September 1, 1908.

GEORGE THOMAS MAWSON, M. Am. Soc. C. E.*

Died October 9, 1928.

George Thomas Mawson was born at Cockermouth, Cumberland, England, on January 14, 1881. Educated at the Carlisle Grammar School, the youth was then articled as a pupil to George Watson and Sons, Architects and Surveyors, Penrith, England, from 1897 to 1900. From this source he received his technical education in the design and construction of public and private buildings and on various municipal undertakings.

In August, 1900, Mr. Mawson went to India to join his brother Mr. E. O. Mawson, of the Public Works Department, where he spent some time on famine relief work in the Rajkot District. He later acted as Personal Assistant to the Under Secretary to the Government Public Works Department of Bombay, on the design and construction of large dams, canals, aqueducts, and irrigation in connection with Government famine relief measures.

In 1903 he joined C. F. Stevens, Architect, in Bombay, as Chief Assistant in charge of the office and superintendence of work. In this position he was engaged on the design and construction of many large banks and churches in that city.

In December, 1903, Mr. Mawson became associated with Messrs. Marsland Price and Company, as head of the Poona Branch. During this service he carried out chiefly Government and public works to the value of $400 000; he held this position until September, 1912, when he was transferred to Bombay, as General Manager, a position he retained until he founded his own firm of Messrs. Mawson, Vernon and Company, in 1919, of which he was Governing Director, until his retirement in June, 1927.

The following is a list of a few of the buildings with which Mr. Mawson was associated in Bombay: The bank building for Messrs. King, King and Company, the Chartered Bank of India, the Mercantile Bank, the Hong Kong and Shanghai Bank, The Times of India, the Church Missionary Society's Building, St. Peter's Church, the British and Foreign Bible Society, the Royal Insurance Company, the Light-foot Refrigerating Company, Limited, and the Free Masons Hall.

In the latter part of his professional career Mr. Mawson was closely identified with reinforced concrete construction, and it may be truly said that he was one of the pioneers of this particular class of work in India. In 1922, he was selected to act on a committee appointed by the Government, which had for its terms of reference,

> "To investigate the utility of Reinforced Concrete for structures in Bombay City by examining such structures as have been in existence for more than 8 years and reporting on their condition, as well as the probable reasons for the corrosion of the steel in any such structures which may be found to be defective."

Thoroughness in all he undertook was the keynote of Mr. Mawson's life. No detail was too small for his personal attention, and clients entrusting work to him, regardless of its nature or size, had every confidence that they were receiving unprejudiced as well as highly technical advice; they felt assured that their interests were safe in Mr. Mawson's hands. He could have extended his business very considerably, but he regulated the number of his contracts so that he could give each one his personal attention and supervision.

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* Memoir prepared by W. Shedden Sinclair, Esq., Bombay, India.

Despite his many professional activities, Mr. Mawson interested himself in all questions relative to engineering. He was a Vice-President of the Bombay Branch of the Institute of Engineers (India), a member of the Bombay Engineering Congress, and was, at all times, ready and willing to give ungrudgingly and cheerfully of his best to the many who sought his aid and advice.

As a member of both English and Scottish Constitutions of Free Masons, he made a wide circle of friends among all classes. He gave much in an unostentatious way to charity and his early death in the prime of life is deeply mourned; India is indeed poorer by the passing of one, who did much toward maintaining and enhancing the highest traditions of the Engineering Profession.

Mr. Mawson is survived by his widow, who was closely identified with his professional career as a Co-Director of his firm, the Mawson Vernon Company, Limited, and with whom much sympathy is felt in her great loss.

Mr. Mawson was elected an Associate Member of the American Society of Civil Engineers on June 30, 1911, and a Member on October 10, 1927.

FRANK MOBERLY, M. Am. Soc. C. E.*

Died July 13, 1928.

Frank Moberly was born in Barrie, Ont., Canada, on July 19, 1845. Educated in both Canada and England, he was a student of law from 1861 until 1863. In 1864 he was the recipient of the First Class Military School Certificate.

Mr. Moberly's first engagement was from 1863 until 1866 in the Engineering Department of the Northern Railway Company of Canada. His work consisted of surveying and similar details; and during the latter part of this engagement he went with the late Sanford Fleming, M. Am. Soc. C. E., as Assistant on Construction of the railway from Picton to Truro, Nova Scotia, Canada. During these years he also served as an Ensign in the Second Administration Battalion on the Niagara Frontier in answer to Canada's first call for volunteers. He was later commissioned as Justice of Peace for the Northwest Territories and the District of Thunder Bay.

In 1867, Mr. Moberly was employed by the Chicago, Rock Island, and Pacific Railway Company on surveys and construction work throughout Western Iowa, but he left this Company in the summer of 1868 to join a location party on the Union Pacific Railway through Utah and Nevada, later being placed in charge of this party. During the winter of 1868 he was transferred to construction work in Weber Canyon, Utah, and remained on this work until its completion in the fall of 1869.

He entered the service of the Milwaukee, Manitowoc, Appleton, and New London Railway Company, in the spring of 1870, in charge of location north of Milwaukee, Wis. He left this position in May of the same year to become Assistant Engineer on exploration of mountain passes for the Northern Pacific Railway Company through the States of Montana, Idaho, and Oregon. Returning to Canada in the fall of 1871, Mr. Moberly was made Assistant on the location of the Muskoka Junction Railway Company from Barrie to Atherly, Ont., Canada, and of the North Gray Railway Company from Collingwood to Meaford, Ont., Canada, leaving that position in May, 1871, to take charge of the first exploration parties for the Canadian Pacific Railway Company from Fort Garry to the Howse and Yellowhead Passes. He also conducted exploration through the territory north of Lac La Bicho and through the Lake Dauphin Country which extended north of the Riding and Duck Mountains, all in Manitoba.

In 1872, he went to Ottawa, Ont., Canada, and accepted the appointment of Assistant to Mr. William Murdoch, of the Canadian Pacific Railway Company, who was outfitting a party to proceed to Fort William, Ont., Canada, to make the first survey and the first location of the Canadian Pacific Railway, from that point westward to Savanna, Ont., Canada, over which the road was subsequently built without alteration. This trip to Fort William was a decidedly adventurous one and entailed a hazardous journey over 200 miles of waterways from Duluth, Minn., down Lake Superior in an open boat. It was at this time that Mr. Moberly received the distinction of being awarded the Dominion Government Medal for the daring rescue of the passengers and the crew from the propeller, *Mary Ward,* which met with disaster in November, 1872, in Georgian Bay.

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* Memoir prepared from information on file at the Headquarters of the Society.

From this time until 1920, Mr. Moberly was active in the following pursuits: In 1875, he was engaged in making an exploration of the country from Gravenhurst to Lake Nipissing for the extension of the Muskoka Junction Railway; in 1876 and 1877, he located and built the Prince Arthur Landing and the Kaministiquia Railway to connect what are now Port Arthur and West Fort William; in 1878, he was Engineer of the Mineral Hill Silver Mine in Nevada; in 1879, Division Engineer with the Canadian Pacific Railway Company surveying and building the line from the mouth of the Nipigon River to the north end of Long Lake and then, in turn, from Long Lake to Missinaibi, Ont., Canada, which was later rebuilt by the Canadian Northern Railway Company; during the summer of 1880 he was engaged on the survey for the Newfoundland Government of a railway line from St. John, New Brunswick, to Harbor Grace, Newfoundland; in 1881 he took charge of the location and construction of the Pambina Mountain Branch of the Canadian Pacific Railway and, subsequently, became Chief Engineer of the Northern Pacific Junction Railway of Ontario; in 1882, he made an exploration, for the Canadian Pacific Railway Company, of the crossing of the South Saskatchewan River at the Elbow and of a line up the Red Deer River; in 1884, he took a contract for the grading of six miles of railroad eastward from the Pic River, which was followed by his accepting the appointment of Commissioner of Police by the Dominion Government, with jurisdiction over Ontario, and his being sent to restore order after the riots in Michipicoton, Ont.; and in 1887 he completed the railroad to Sault Ste. Marie, Ont., Canada.

In 1891, and extending over a period of five years, Mr. Moberly was engaged in various private undertakings. In addition, he made a survey of Collingwood Harbor for the Department of Railways and Canals of Canada, and was active also in the construction of the St. Lawrence and Adirondack Railway, as well as surveying and promoting an air line from Toronto, Ont., to Collingwood; in 1896, he went to Rossland, B. C., Canada, where he was engaged in prospecting east and west of Kootenay, and in part of the Yale Districts, as well as on the construction of Red Mountain Railway into Rossland, and on the location of the Crows Nest Pass Railway, along the shore of Kootenay Lake. In 1900 he was employed with the Department of Lands and Works of British Columbia, having charge of one of two parties sent out by the Government to make surveys of passes through the Hope Mountains, which, to that time, had been considered impracticable for a railroad; in 1902, he made estimates for such a passage through these mountains and their boundary country and also plans for a railway to Dease Lake in Northern British Columbia; in 1903, he had charge of the construction of the Prince Albert Branch of the Canadian Northern Railway, from Erwood to Melfort, Sask., Canada; from 1904 to 1909, he was in charge of party on preliminary and location surveys for the National Transcontinental Railway Company, east of Cochrane, Ont., Canada, and along the north shore of Lake Abittibi extending eastward to the Harricanaw River; and from 1910 to 1912, he made explorations and reports on the capabilities of the great clay belt for the Chairman of the Temiskaming and Northern Ontario Railway Company.

In 1913, Mr. Moberly was appointed Assistant Engineer of the Department of Public Works of Canada, for which he had charge of the territory in the Midland District and also of that around Toronto. He later was made Acting District Engineer at Midland, and Barrie, Ont., which latter place he made his home after his retirement. Altogether, Mr. Moberly was in the service of the Government of Canada for about twenty-three years which period terminated in 1920, when he retired from active work. During this time he had practically explored the Dominion from coast to coast, opening it up for settlement, and making way for its widely expanding civilization.

Coupled with Mr. Moberly's professional services and experiences during the period extending from 1880 to 1915 was a definite and continuous service on his part in civil duty to the State. The following includes some of his positions in this connection: Provincial Magistrate for the Province of Manitoba; Reeve of Neebin Municipality, District of Thunder Bay; Commissioner of Police under the Police Act of Canada for Ontario; and Commissioner under Peace Preservation on Public Works Act; Justice of Peace for the Province of British Columbia; Magistrate for the Province of Quebec; Police Magistrate for the District of Nipissing; Justice of Peace for the County of Simcoe; and President of the County of Simcoe Home Guard Rifle Association.

In 1874, Mr. Moberly was married to G. A. McIntyre, the daughter of John McIntyre, a Hudson Bay Officer at Fort William, Ont. Mrs. Moberly died in 1880, and, in 1882, Mr. Moberly was married to Mary McIntyre, a sister of his first wife, by whom, with two daughters and two sons, he is survived.

Mr. Moberly's entire life is an expression of pioneering in Canada when the Dominion's future was much less assured than it is to-day. To have been an explorer all his days, in itself, lends a glory and a romance to his name. He was a pioneer in the greater sense of the word, in that he did not settle on a small clearing of land and thereby establish a permanent comfortable home, but was ever blazing new trails in virgin country—leaving the secure and certain to encounter the risk of the unknown. He surveyed railways, hunted criminals, started clearings, dug mine shafts, and endured the prairie winter in tents, ever measuring the uncalculated and keeping continuously in the forefront of civilization; this was his life. John F. Stevens, Past-President, Am. Soc. C. E., extends to Mr. Moberly and his associates the following tribute:

> "The railways—indeed, civilization itself—owes a debt of gratitude to the pioneer engineers who blazed the way through trackless wildernesses to make homes for thousands who followed them."

Mr. Moberly was elected a Member of the American Society of Civil Engineers on October 7, 1903.

ALEXANDER BAIN MONCRIEFF, M. Am. Soc. C. E.*

DIED APRIL 11, 1928.

Alexander Bain Moncrieff was born in Dublin, Ireland, on May 22, 1845, a descendant of an old Scottish family of Perth which numbers among its members a noted lawyer, a famous divine, and many others distinguished in military and civil affairs.

Mr. Moncrieff spent several years at the Belfast Academy, after which he was articled to the late Mr. E. Miller, Chief Engineer of the Great Southwestern Railways in Ireland. He served his apprenticeship in the railway shops at Inchicore, Dublin, and had seven years' experience in the workshops and in the office of the same Company. This period included twelve months spent in the blacksmith's shop, where he worked from 6:00 A. M. until 5:00 P. M., in close contact with a side of life that developed in him a feeling of keen sympathy for his fellow workers, to whom he always said he owed a great deal.

At the close of this training and experience Mr. Moncrieff went to Glasgow, Scotland, as Draftsman for the Dubbs Works, at Govan, where he was engaged mainly on the design and construction of locomotives. Because of business changes, he returned to Ireland and took charge of the rebuilding of a large water-driven flour-mill and malthouse at Milford-on-Barrow. On the completion of this work, Mr. Moncrieff assumed charge of a small workshop in London, England. After a short interval, however, he answered an advertisement issued by the South Australian Government for an efficient Draftsman. His application met with immediate success and he accordingly took ship to Australia, where he arrived in February, 1875. As Engineering Draftsman he was chosen to assist in the design of fortifications and in this capacity he was obliged to travel to Melbourne, Sydney, and Brisbane, Australia.

In 1879, Mr. Moncrieff was appointed Resident Engineer of the South Australian Railways, taking charge of the line as it was gradually extended from Port Augusta to Oodnadatta. In 1888, on the retirement of the late H. C. Mais, M. Am. Soc. C. E., he was promoted to the office of Engineer-in-Chief, which position he held for twenty years. In addition to his duties in the Railways Department, he had charge of the construction and maintenance of harbors, jetties, and lighthouses, as well as the construction of water-works and the conservation of water. The construction of the Happy Valley Water Works and of the Outer Harbor, Adelaide, are two outstanding monuments to his ability, determination, and zeal—true evidences of engineering achievement of which any engineer might justly be proud. They stand as silent memorials to a career of exceptional usefulness and conscientious public service.

On the retirement of the late Mr. Alan G. Pendleton, Mr. Moncrieff succeeded to the office of Railways Commissioner. As such, he served under nineteen Commissioners of Public Works, from all of whom he received hearty support in his work. Under Mr. Moncrieff's supervision many improvements were accomplished, including the completion and putting into service of the

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* Memoir prepared from information furnished by Alex. S. Moncrieff, Esq., Adelaide, South Australia.

Mile-End Goods Yard, by which means up-to-date freight accommodation was provided for the City of Adelaide and its suburbs; new siding accommodations at Port Adelaide; the installation of appliances for the prevention of fire at railway stations; the improvement of sanitary arrangements at stations and of the facilities on the Woodville and Henley Beach Lines; the increase in and the improvement of the rolling stock of the railways; the fitting of the Westinghouse brake to a large number of freight trucks, which made possible the delivery of parcels in the suburban area by motor vehicles; the establishment of an inquiry office at the Adelaide Railway Station (an opportunity was also given the apprentices to improve themselves by attending classes at the School of Mines); the placing of railway fares on a scientific basis; the teaching of first aid to a large number of the staff and the planting of gardens at stations which was encouraged wherever possible. Mr. Moncrieff also was the "father" of the South-Eastern Drainage Project in which he took great interest and which at its inception he personally explained to conferences of landholders held at Narracoorte and Millicent, Australia, ensuring their hearty support of his proposals.

In 1916, after forty-two years of service in several capacities, Mr. Moncrieff retired as Railways Commissioner. During this entire period his only vacations were those spent in traveling on Departmental business. When asked once by a representative why he did not take a holiday he replied, "I do not feel that I need a holiday and the fact that I have never been ill proves it." Mr. Moncrieff was a living justification of the gospel of hard work.

He was the first Chairman of the Municipal Tramways Trust and held this position from its inception in 1907 until January, 1922. He was a member of the Institution of Civil Engineers of Great Britain, and in December, 1909, under the hands of the King of England, was made a Commander of the Order of St. Michael and St. George.

Until shortly before his death, Mr. Moncrieff had enjoyed the most robust health, taking a constitutional walk to and from the city several times a week. Also, before his retirement, he walked daily to his office from his home. He was a lay reader of the Church of England, and was greatly interested in church work. Gardening, reading, mechanics, and Free Masonry were also among his recreational pursuits and in these releases from his strenuous official career he found joy and relaxation.

Mr. Moncrieff was a man of decided opinions and of great personal force. The Secretary to the Railways Commissioner, Mr. C. J. Boykett, in referring to Mr. Moncrieff, has stated that officers who were intimately associated with him in practically all the positions he occupied testify to his outstanding capability, versatility, and energy displayed in carrying out his undertakings. He was held in high esteem for his gentlemanly and honorable conduct by all those with whom he was associated in the railway service, and was regarded with warm affection by those with whom he was more intimately acquainted. A true gentleman, kindly and most gracious by nature, his passing is mourned by a great company of associates and friends. He is survived by his widow, Mary Bronson Moncrieff, a son, and a daughter.

Mr. Moncrieff was elected a Member of the American Society of Civil Engineers on July 4, 1894.

FRED FORREST MOORE, M. Am. Soc. C. E.*

Died June 2, 1928.

Fred Forrest Moore was born in Waltham, Mass., on October 8, 1867, the son of Edwin F. Moore and Mary E. (Small) Moore. He received a part of his education in the public schools, leaving High School in order to prepare himself for the Massachusetts Institute of Technology, from which he was graduated in 1891 with the degree of Bachelor of Science in Civil Engineering. While he was attending college, his work attracted the attention of President Amasa D. Walker, who invited him to join the Teaching Staff, but Mr. Moore preferred the practice of his profession.

Previous to entering college, he had had considerable experience in surveying, and in 1890 and 1891, having been granted a leave of absence during the Institute term, he was in charge of a plane-table party on topographical surveys of lighthouse reservations along the Maine Coast. Directly upon graduating from the Institute, he joined the Engineering Force of the Boston Water-Works, for which he was engaged on the design of the Sudbury Dam and Reservoir. In 1896, this work was taken over by the Massachusetts Metropolitan Water Board, and Mr. Moore continued in charge of designs and estimates of the Headquarters Field Office. On the completion of this work, he became Principal Assistant to the Superintendent of the Sudbury Department with maintenance and operation duties.

From October, 1900, to April, 1902, as Assistant Engineer on filtration experiments for Springfield, Mass., Mr. Moore constructed plant and carried on experiments on slow sand filtration under the joint direction of the Massachusetts State Board of Health and the Consulting Engineer. From April, 1902, to February, 1903, he was engaged on designs for a concrete railway bridge and other structures connected with the Wachusett Reservoir built by the Massachusetts Metropolitan Water and Sewerage Board.

From February, 1903, to January, 1904, he served as Principal Assistant Engineer on studies, designs, and estimates for the Aqueduct Department of the Burr-Hering-Freeman Commission on Additional Water Supply for the City of New York. During the next three months, Mr. Moore held a similar position with the Northern New Jersey Flood Commission. From April, 1904, to September, 1905, he was Office Engineer, and from the latter date to April, 1906, Hydraulic Engineer, for the Committee of Twenty of the National Board of Fire Underwriters.

From April, 1906, until his retirement on March 2, 1928, Mr. Moore was a Designing Engineer for the Board of Water Supply of New York City. In this position he was in charge, under the Department Engineer, of studies and designs for cut-and-cover aqueducts, dams, and reservoirs, including landscape work, steel pipe siphons, distribution lines, and sewage disposal works.

He was a member of the Municipal Engineers of The City of New York, the New England Water Works Association, the Massachusetts Institute of

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* Memoir prepared by Franklin H. Robbins, M. Am. Soc. C. E.

Technology Alumni Association, the Masonic Fraternity, the Royal Arcanum, the Arkwright Club, and the Nannahagan Golf Club.

Mr. Moore was admired by his many friends for his kindly disposition, his fair dealing, and for the ability and energy which he devoted to his professional work. On October 5, 1896, he was married to Winnefred T. Wright, who survives him.

Mr. Moore was elected a Member of the American Society of Civil Engineers on April 3, 1907.

ARCHY MAGILL NELSON, M. Am. Soc. C. E.*

DIED JUNE 26, 1929.

Archy Magill Nelson was born at Green Mountain, Va., on November 1, 1862, the son of Archie M. and Eliza Price Nelson. He received his education at Washington and Lee University, Lexington, Va.

Mr. Nelson's early engineering experience was acquired in service with the Pennsylvania Railroad Company and on the Schuylkill Valley Railroad from 1882 to 1885, where he advanced rapidly from Axeman to Assistant Engineer in charge of construction and location. He remained with the Pennsylvania Railroad Company until 1890 in charge of heavy construction and difficult location work.

In succeeding years he was employed in the following positions. From 1890 to 1900, as Principal Assistant Engineer on the Kansas City Southern Railroad, having charge of location and construction of approximately 500 miles of that railway, including yard and terminal and division point layouts and facilities; from 1900 to 1908, with the Kansas City, Mexico and Orient Railroad Company, in charge of very heavy and difficult location in Old Mexico (the Sierra Madre Mountains), and of construction between Chihuahua, Mexico, and the Pacific; from 1909 to 1911, with the Oregon Short Line Railway Company, in charge of a resurvey of about 2 500 miles of line, and, later, as Valuation Engineer on the Oregon Short Line and Southern Pacific Lines East of Sparks, Nev. From 1916 to 1919, he served as State Highway Engineer of Tennessee.

Mr. Nelson was prohibited from active service in the World War on account of his age, despite the fact that his services were freely offered.

In 1919, following this period, he went to Memphis, Tenn., as County Engineer for Shelby County, acting as such until the fall of 1926. In addition to the opening, renewal, maintenance, etc., of many miles of county highway, with accompanying bridges, culverts, etc., Mr. Nelson designed and built numerous plants for the storage of gasoline and the storage and treatment of oils and other ingredients used in the carpet treatment of highways. A noteworthy piece of work was an 150-ft. reinforced concrete bridge over Nonconnah Creek on the Hernando Road.

In 1926, Mr. Nelson had planned to retire from active practice, but rather reluctantly consented to plan and build twin tunnels through Missionary Ridge, for Hamilton County, Tennessee. These tunnels were each more than 1 000 ft. long, and of concrete construction. They were built for highway purposes and to develop suburban Chattanooga, and to connect that city more directly with Atlanta, Ga. The cost of the tunnels was approximately $500 000. Mr. Nelson's death occurred just before the finishing touches were put on the work.

In 1909, he was married to Lucy Beesley, of Murfreesboro, Tenn., who survives him.

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* Memoir prepared by M. P. Paret, M. Am. Soc. C. E.

A member of the Protestant Episcopal Church he was much interested in later years in church work in Memphis. In his life and work he was intensely practical, his engineering experience and knowledge being largely acquired in the "University of Hard-knocks". He had an uncanny ability to meet emergencies successfully.

He was a devoted husband and a loyal friend, always considerate of and highly respected by his associates and employees.

Mr. Nelson was elected a Member of the American Society of Civil Engineers on October 4, 1910.

ROBERT VAN ARSDALE NORRIS, M. Am. Soc. C. E.*

Died April 20, 1928.

Robert Van Arsdale Norris, the only son of Thomas Braxton and Mary Latimer (Ruxton) Norris, was born in Newark, N. J., on May 2, 1864. He came of distinguished New England ancestry, being directly descended from Governor William Bradford, John Alden, and Timothy Dwight.

After a preparatory course in the Collegiate School of New York, N. Y., Mr. Norris entered the Columbia University School of Mines, and was graduated with the degree of Mining Engineer in the Class of 1885, a class that included in its membership a number of men who became distinguished in their professions. From among them, young Norris was selected as Assistant to Professor H. C. Munroe in the Summer School of Mining and Surveying, which engagement followed his graduation. In the latter part of 1886, after he had served a short time as an Inspector of dredging operations on the Maurice River, and in the Chemical Laboratory of Herman Behr, in Brooklyn, N. Y., he joined the Engineering Corps of the Susquehanna Coal Company, then owned by the Pennsylvania Railroad Company. Here he rose through sheer ability and hard work to the position of Chief Engineer in 1900, remaining until 1904, when he resigned in order to engage in private business as a Consulting Engineer. His judgment in making the venture proved sound, for he was eminently successful. Early in 1923 Mr. Norris took his son, Robert V. Norris, Jr., into partnership with him, under the firm name of R. V. Norris and Son.

That the exercise of sound judgment was such a characteristic of Mr. Norris as to pertain to the nature of a habit is attested by the fact that he was retained as Consulting Engineer by the Coal Department of the Delaware, Lackawanna and Western Railroad Company (afterward becoming an independent organization incorporated as the Glen Alden Coal Company), the Lehigh Valley Coal Company, Coxe Brothers and Company, Incorporated, the Coal Department of the Pennsylvania Railroad Company, the Wilkes-Barre and Hazleton Railroad Company, the Lehigh Coal and Navigation Company, the Sterling Salt Company, and other important interests.

Mr. Norris specialized in the examination and valuation of coal properties, his most important undertaking of this nature being probably the appraisal of the anthracite properties of the Pennsylvania Railroad Company, which eventuated in their sale, in 1915, to M. A. Hanna and Company, of Cleveland, Ohio, and the incorporation of the Susquehanna Collieries Company as the operating company for the Hanna anthracite properties. The year previous (1914), Mr. Norris had made a valuation report on six collieries of the Temple Coal Company. Other major, or valuation, reports made by Mr. Norris were on the properties of the Lehigh Coal and Navigation Company, in 1917; the bituminous coal properties of the Stonega Coal and Coke Company of Virginia and the Consolidation Coal Company of Maryland, in 1920; the bituminous coal lands of the Cambria Steel Company, in 1922; the anthracite properties of Madeira, Hill and Company and of the Hazle Brook Coal Company, in 1923; the anthracite properties of the Hudson Coal Company, Delaware and Hudson Company, Delaware, Lackawanna and Western Railroad Company, and Lehigh Valley Coal Company, in 1924; Coxe Brothers and Company, Incorporated, the Scranton Coal Company, and the Elk Hill Coal and Iron Company, in 1925; and the Jeddo-Highland Coal Company, in 1926. He also served as arbitrator in many cases involving complicated technical problems.

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* Memoir prepared by Edward W. Parker, Esq., Philadelphia, Pa.

Mr. Norris was not only an engineer; he had a special gift in cost accounting, and, in co-operation with Col. R. H. Montgomery, of Lybrand, Ross Brothers and Montgomery, prepared a system of cost accounting for anthracite operators. His facility with figures approached the uncanny. It took him but a short time to familiarize himself with the fundamentals of the Federal Income Tax laws as applied to the involved technicalities of anthracite production, and he served as an expert for a number of companies before the Tax Authorities, in Washington, D. C.

During the World War, Mr. Norris was called to Washington to serve as an adviser to the War Industries Board on fixing prices for anthracite and bituminous coal. Later, in association with Cyrus Garnsey, Jr., and James H. Allport, M. Am. Soc. C. E., he served as a member of the Engineers' Committee of the United States Fuel Administration. He became "a-dollar-a-year" man and, giving up his own practice, rented a house in Washington and moved his family there. His two daughters, then unmarried, served as ambulance drivers.

In his busy career Mr. Norris found time to give to others the benefit of his own knowledge and experience. At various times he was a non-resident lecturer on coal mining, both at his Alma Mater, Columbia, and at Harvard Universities. The *Transactions* of the American Institute of Mining and Metallurgical Engineers have been enriched by many of his contributions. He was a Councillor of the Institute from 1908 to 1910, Vice-President in 1911 and 1912, and a Director from 1920 to 1927. He served on numerous committees at various times and was ever enthusiastic and tireless in his devotions to that which he felt was for the best interests of the organization.

In 1914, Columbia University conferred upon Mr. Norris the honorary degree of Master of Science and in 1920 he was the '89 Medallist. He was always active in the Alumni affairs of the University. In addition to membership in the American Institute of Mining and Metallurgical Engineers, Mr. Norris was a member of the American Society of Mechanical Engineers, and of the Canadian Institute of Mining and Metallurgy. He was a member of a number of clubs, which included the University and Engineers' Clubs of New York, and the Cosmos and Chevy Chase Clubs of Washington.

He is survived by his widow, Esther Wadhams (Shoemaker) Norris, one son, Robert V. Norris, Jr., of Wilkes-Barre, and two daughters, Mrs. Jane N. Zerbey, of Pelham, N. Y., and Mrs. Esther S. Triest, of New York.

It falls to the lot of relatively few men to attain so eminent a position in their profession as that achieved by "Van" Norris in his chosen profession—mining engineering, specializing in anthracite mining.

At the time of his death, and for some years prior thereto, Mr. Norris was generally recognized as the dean of the profession in the anthracite region. When important problems were to be solved, were it the fighting of a bad mine fire, like the one at the Woodward Colliery of the Glen Alden Coal Company; the valuation of coal-mining properties for purposes of sale, or for other reasons, such as that of the Susquehanna Collieries Company by the Pennsylvania Railroad Company to the M. A. Hanna interests; the arbitration of a serious dispute involving engineering problems; the adjudicating of complicated income tax matters with the Federal authorities, or ordinary tax controversies with the State and local officials—whatever the difficulty—Mr. Norris was the one selected for the job. His unimpeachable integrity, scientific training, wide experience, and sound judgment commanded the confidence of those in whose interests he served as well as the respect of his opponents.

A brief sketch of Mr. Norris' life published in a technical journal,* closes with the following words:

> "An engineer of the highest rank, a man of the most upright character and of marked personality, he was held in high esteem by all who knew him, and his death at an age when he apparently had many more years of active public service before him will be deeply mourned."

This is indeed true; he is deeply mourned, and his profession lost a brilliant disciple when he passed away.

Mr. Norris was elected a Junior of the American Society of Civil Engineers on December 7, 1887, and a Member on March 5, 1902.

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* *Mining and Metallurgy*, May, 1928.

EBEN ERSKINE OLCOTT, M. Am. Soc. C. E.*

Died June 5, 1929.

Eben Erskine Olcott was born in New York, N. Y., on March 11, 1854. He was the second son of a family of four sons and four daughters, the children of John N. and Euphemia Helen (Knox) Olcott.

After a preparatory training in the College of the City of New York, where his early interest in scientific and technical subjects manifested itself, Mr. Olcott entered the Columbia School of Mines, and was graduated therefrom in 1874 with the degree of Engineer of Mines. As a student, he stood high in a class that included among its members a group of young men who subsequently made their mark in professional life.

Immediately after graduation, Mr. Olcott began his field career as Chemist to the Ore Knob Copper Company, of North Carolina, where he was soon put in charge of the then well-known Hunt and Douglas process of copper extraction. In 1875, he was appointed Assistant Superintendent of the Pennsylvania Lead Company, Mansfield Valley, Pa. In 1876, he went to South America for the Orinoco Exploring and Mining Company, of Venezuela, and in 1878-79 served as Superintendent of the gold mines of this Company, near El Callao, Venezuela. Returning to the United States in the latter part of 1879, he spent two years in examining and reporting on mines in a number of the Western States. From 1881 to 1885 he was Superintendent of the St. Helena Gold Mining Company, Sonora, Mexico. On resigning this post, after four years of strenuous work under untoward conditions in the development of the property and erection of plant, he took up consulting practice, with headquarters in New York.

In 1886, Mr. Olcott was engaged for ten months in an elaborate examination of the Cerro de Pasco Copper District, in Peru. At the same time Mr. A. D. Hodges, Jr., a prominent mining engineer of Boston, Mass., made an independent reconnaissance. Their combined studies and recommendations furnished the basis upon which were founded, years afterward, the extensive operations of the present Cerro de Pasco Copper Company. Next, at the instance of groups of New York capitalists, Mr. Olcott headed two protracted prospecting expeditions to the gold fields of Dutch and British Guiana, and another to the Provinces of Tolima and Antioquia, in the Republic of Colombia.

In 1890-91, for the Peruvian Exploration Syndicate, of London, England, he examined and reported upon the gold and copper resources of the Sandia and Carabaya Districts of Eastern Peru. This was a task involving considerable difficulty and hardship, on account of the remoteness of the very sparsely populated regions traversed, the entire lack of modern means of transport, and the great altitude (reaching at least 17 000 ft.) of some of the terrain. After leaving the railway, south of Cuzco, the entire journey, occupying a number of months, was made on mule-back or on foot. Thereafter, for about ten years, he was actively engaged in professional work in Mexico, British Columbia, Ontario, New Brunswick, and various parts of the United States.

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* Memoir prepared by Prof. Robert Peele, New York, N. Y.

Mr. Olcott was a member of the American Institute of Mining and Metallurgical Engineers almost from its inception, having been elected in 1873 (while still a student), in the third year of the life of the Institute. For two years (1901-02) he served as its President, and, in 1901, attended a meeting of that body held in Mexico, delivering a Presidential Address in the City of Mexico, in the Spanish language, before a representative assembly of State officials and engineers.

Mr. Olcott's professional and business life of fifty-four years was rather sharply divided into two distinct periods. For the first twenty-one years he devoted himself solely to mining engineering and metallurgy, as outlined previously. Then, circumstances led him into a wholly new sphere of activity, very distantly related to the profession in which he had been trained by faithful study and long experience.

Following the death of his brother-in-law, Charles T. Van Santvoord, General Manager of the Hudson River Day Line, Mr. Olcott in 1895 accepted an appointment to the managership of that line of passenger steamers, running between New York and Albany, N. Y., which had been founded by his father-in-law, Commodore Alfred Van Santvoord. On the death of the latter, in 1901, Mr. Olcott was elected President and General Manager of the Company, which at that time was operating two steamers, the *Albany* and the *New York*. In the succeeding years, as traffic increased, new steamers were added, until at his death the fleet comprised the *Hendrick Hudson, Alexander Hamilton, Robert Fulton, DeWitt Clinton, Peter Stuyvesant, Albany* and *Chauncey M. Depew.*

In his management of the Day Line, Mr. Olcott exhibited his characteristic energy and resourcefulness. No detail that might contribute to the success of the business was too small to receive his personal attention. He made himself familiar with every feature of the boats, from the boiler and engine rooms to the Commissariat Department. His hours of work during the operating seasons were much longer than those of any employee. Unquestionably, his engineering training aided materially in achieving the success attending his control of the Company's large business.

During this second period of his career, he continued for a number of years to give part of his time, in an advisory capacity, to the affairs of the consulting engineering firm of Olcott, Fearn, and Peele, which was organized in 1896. Later (1901), the firm became Olcott, Corning, and Peele, and, finally, Olcott and Corning.

Mr. Olcott had a forceful, impressive personality. Among his outstanding characteristics were his integrity, large humanity, kindly courtesy, and never-failing helpfulness. These qualities secured the affection, loyalty, and respect of all who were brought into contact with him—employees and business associates alike.

His wide business and social interests and his sympathetic nature are indicated by the diversity of the large number of philanthropic, religious, and other organizations with which he was connected in various capacities. Of these, only a comparatively few can be mentioned here: Trustee of the American Seamen's Friend Society, and Treasurer and Trustee of the American Indian Institute; Vice-President of the Lincoln Safe Deposit Company and of the Evangelistic Committee of New York City; member of the Administration Committee of the Federal Council of the Church of Christ in America; member of the Advisory Committee, American Exchange-Irving Trust Company and of the Travelers' Aid Society; member of the Board of Managers, American Bible Society; member of the New York Chamber of Commerce, Merchants' Association, Metropolitan Museum of Art, New York Historical Society, Sons of the Revolution, the American and New York State Forestry Associations, National Board of Steam Navigation, and the Board of Foreign Missions of the Dutch Reformed Church of America. He was also a member of the following New York clubs: Union League, Engineers', and the Down Town Association.

Mr. Olcott was married in 1884 to Kate Van Santvoord who, with three sons and one daughter, survives him. He died on June 5, 1929, after a long illness. His eldest son, Alfred, for years associated with him in the conduct of the Hudson River Day Line, has latterly been Vice-President and General Manager of the Line. One of his brothers, Judge William M. K. Olcott, was formerly District Attorney of New York.

Mr. Olcott was elected a Member of the American Society of Civil Engineers on July 5, 1893.

ROBERT STEVENS PARSONS, M. Am. Soc. C. E.*

Died May 18, 1928.

Robert Stevens Parsons, the son of the Rev. Solomon and Louise (Towt) Parsons, was born in Hohokus, N. J., on May 26, 1873. He was prepared in Paterson High School for Rutgers University at New Brunswick, N. J., which he entered with a State Scholarship and from which he was graduated in 1895 with a degree of Bachelor of Science. Three years later he received the degree of Civil Engineer. As an undergraduate, Mr. Parsons' life was a most strenuous one and his days were occupied with numerous campus activities. He enjoyed the honor of being Captain in the Military Corps and of the football and track teams, as well as the further distinction of being a member of the Delta Upsilon Fraternity, and President of the Self-Governing Board and of his Senior Class.

Within a week of his graduation, Mr. Parsons entered the employ of the Erie Railroad Company as Chainman. From the very bottom rung of the engineering ladder, he worked up through all departments until he attained the position of Chief of Maintenance of Way, from which he transferred to the Operating Department as Superintendent. In this capacity, he greatly distinguished himself by handling a difficult strike. Continuing his upward climb he reached the position of Chief Engineer and, at length, Vice-President in charge of maintenance and operation.

Mr. Parsons was a Trustee of Rutgers University and one of a group that encouraged and strengthened athletic activities at the time when the Rutgers football eleven was greatly lauded among college teams. He was very active in civic work in the communities in which he resided, and during his term as Vice-President, when one of his duties was the building up of friendly relations between the communities and the railroad, became a well-known public speaker.

He was a member of the New York Railroad Club, the American Railway Engineering and Maintenance of Way Associations, and the New Jersey Highway Commission, and was affiliated with the Masonic Lodge at Nutley, N. J., and the Rotary Club. He was also a Lieutenant in the New Jersey Militia and an Officer of the Military Reserve.

In 1897, Mr. Parsons was married to Eleanor Howse, of Paterson, N. J., and is survived by his widow and two daughters, Mrs. Robert Miller, of Nutley, and Mrs. Asel Adams, of Youngstown, Ohio.

"R. S.", as he was affectionately known, had a strong, dominant personality, and an army of friends ranging from the humblest trackmen to distinguished railroad executives. He was always cheerful, a marked optimist, and although frequently under the severest of strains, he was able to cheer and stimulate his subordinates. The following revealing remark comes from one of them: "How can we crab when 'R. S.' is around?"

It is remarkable that with this friendly relationship among his associates there occurred no decrease in efficiency and co-operation, these being two prime essentials that Mr. Parsons always demanded. He was a real man's man, loved by thousands, and his services were invaluable to his Company. In the words of the First Vice-President of the road is expressed the thought of his many friends: "I do not see how we are going to find one to replace him".

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* Memoir prepared by Floyd Y. Parsons, Esq., New York, N. Y.

Mr. Parsons was elected a Member of the American Society of Civil Engineers on September 6, 1905.

L. FREDERICK PATSTONE, M. Am. Soc. C. E.*

Died July 13, 1927.

L. Frederick Patstone was born on June 24, 1876, at Providence, R. I. He attended the Providence Manual Training High School, and from 1895 to 1899 was a student in the office of the City Engineer of Providence. During this period he also studied at the Rhode Island School of Design and took a special course in Civil Engineering under John E. Hill, M. Am. Soc. C. E., at Brown University. He later received the degree of Civil Engineer from Ohio Northern University.

From June, 1899, to February, 1903, Mr. Patstone served as Assistant Engineer in the City Engineer's Office of Providence. Having taken and successfully passed a Civil Service examination, he received a Government appointment as Civil Engineer in the Philippine Islands, which service he began on March 3, 1903. Thereafter, for the remainder of his life with the exception of a brief absence during the World War, he was identified with engineering activities in the Islands and in China.

The early part of this work was in the Department of Engineering and Public Works at Manila. Successively, except for the two years, 1914 to 1916, when he was in private practice as Civil Engineering Contractor, and July, 1918, to July, 1919, when he was an officer in the World War, he was Superintendent of Streets and Bridges, First Assistant to the City Engineer, Acting City Engineer, and City Engineer of Manila. He also served as Superintendent of Water Supply and Sewers, and Acting Mayor of Manila.

To recount all the accomplishments under Mr. Patstone's direction is unnecessary; a mere general review of the different kinds of work included will give an idea of the extent and detail involved. He had charge of laying thousands of square yards of cement sidewalk and pavement. Various bridges were constructed under his supervision, including reinforced concrete arches, several highway bridges of more than 200 ft. span, and a steel lift bridge. A large stone-crushing plant, barges for handling the crushed stone, reinforced concrete school buildings, municipal markets, asphalt and macadam roads, a 3 000-ft. sea-wall—these brief items indicate the wide variety of his work.

Early in his stay in Manila he evolved a city plan covering an area of 68 sq. miles. It was his pleasure to see this plan in part developed under his supervision. Many miles of road were thus built. Much water-front land was reclaimed for developing the harbor. The extensive drainage, construction, and sanitary improvements were incidental to this development. Finally, during the latter years of his régime, he was privileged to perfect many of the plans previously evolved, for the future development of this important city.

During the World War, Mr. Patstone, already a Major in the Officers' Reserve Corps of the Army of the United States, was detailed as Colonel and Chief of Engineers of the Philippine National Guard, but he resigned from this position as soon as permission could be obtained, to enter the Engineer

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* Memoir prepared by A. Gideon, M. Am. Soc. C. E.

Officers' Training Camp at Camp A. A. Humphreys, Va., after which he served as Major of the 217th Engineers, U. S. Army.

In February, 1919, he returned to Manila to resume the office of City Engineer. This position, however, he resigned the following year in order to engage in a commercial interprise in Shanghai, China. He was Vice-President of the Asia Development Company, with headquarters in Shanghai, and personally directed a notable engineering work carried on by the Company in Shantung Province in 1922 and 1923 when a vast area of land was reclaimed and saved from the flood waters of the Yellow River by extensive diking. He afterward became a member of the firm of Wooten and Patstone, Architects, in Shanghai, and was thus engaged when he was taken with his last sickness. He died at the Shanghai General Hospital on July 13, 1927, after an illness of three months.

Mr. Patstone was a member of the following technical societies and clubs: The Boston Society of Civil Engineers; the Elks, and the Army and Navy Clubs, of Manila; the French, American, Union, and Columbia Country Clubs, and the Chinese-American Association of Engineers, all of Shanghai.

In addition, he was also a member of the American Chamber of Commerce of Shanghai, for which he served as Chairman on the Committees of Architects and Engineers. He was a Member of the Benevolent and Protective Order of Elks and a Thirty-second Degree Mason, Member of Afifi Temple, A. O. N. M. S., Member of Bamboo Oasis of Manila, and President of the Nomad Oasis in Shanghai. He served as a Member of the Traffic Commission of Shanghai, and, after that body was dissolved, he became a Land Commissioner, having been a nominee of the Shanghai Municipal Council for that position. He also served with the Shanghai Volunteer Corps, as a member of the American Mounted Troop.

As Mr. Patstone devoted the best years of his professional life to the Manila service, it is there that he was best known and best appreciated for his true worth. His success was due not only to his technical and administrative ability, but in no small measure to his sympathetic appreciation of the real mission of the American Government employee in the Philippines: "To train the man under you to fill your position". Certainly he did that; he trained a Filipino as his own successor as City Engineer of Manila. Not until, himself at the top, he saw the City Service practically Filipinized, did he feel free to seek the greater financial gain which private practice had to offer.

In addition to his technical ability, Mr. Patstone was an accomplished musician and was endowed with a remarkable singing voice. He was always ready to use this talent for charity concerts and the pleasure of his friends.

He was married on March 21, 1908, at Calapan, Mindoro, Philippine Islands, to Daisy Suter Clark, the daughter of Captain J. J. Clark, U. S. Army, who survives him.

Mr. Patstone was elected a Member of the American Society of Civil Engineers on April 2, 1913.

YSIDORO YGNACIO POLLEDO y SANDS, M. Am. Soc. C. E.*

DIED MAY 7, 1928.

Ysidoro Ygnacio Polledo y Sands was born in Matanzas, Cuba, on July 31, 1865. A student in his native city, as well as in Germany and France, he also attended the School of Mines at Columbia University, in New York City, and was graduated with the degree of Mining Engineer in 1885.

Following his graduation, Mr. Polledo served as Assistant Engineer on the survey for a water-works for the City of Santiago de Cuba, Cuba. In 1886, he was appointed Assistant Engineer, and, later, Principal Assistant Engineer, in charge of track and structures for the Cárdenas and Júcara Railroad Company, Cárdenas, Cuba. From 1889 until 1890 he acted as Manager of the Santa Bárbara Plantation and Sugar Factory, at Baró, Cuba, later becoming General Manager for the Cárdenas Sugar Refining Company, at Cárdenas.

In 1895, Mr. Polledo again varied his field of occupation, when he was appointed Assistant Engineer on the Harbor Works of Havana, Cuba, which position he held until the following year, when he engaged in private practice as a Consulting Engineer. In this capacity, he re-designed sugar houses, and the evaporating, condensing, and water-cooling plants for sugar factories in Santa Bárbara, Elizalde Santa Catalina, Alava, La Luisa, Carmen de Crespo, and Narcisa, Cuba.

In 1899 he again became Assistant Engineer on the Harbor Works at Havana, under the United States Military Government, Division of Cuba, Department of Havana. During subsequent years, until 1920, Mr. Polledo was employed in the following positions: Technical Director of the Santa Lucia Sugar Factory, Gibara, Cuba; General Manager of the Matanzas Railroad Company; Manager of the "Banco Español de la Isla de Cuba"; Consulting Engineer in private practice and Vice-President of the Ferro-carril del Norte de Cuba. From 1920 until his death, he served as Vice-President of the Regla Coal and Oil Company.

Mr. Polledo was a man of strict integrity which was applied in his profession as well as in his public life. He is survived by two daughters.

He was a member of the Cuban Society of Engineers, the American Institute of Mining and Metallurgical Engineers, and the American Society of Mechanical Engineers.

Mr. Polledo was elected a Junior of the American Society of Civil Engineers on January 2, 1889, and a Member on November 4, 1903.

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* Memoir prepared by Dra. Cira Polledo, Matanzas, Cuba.

CHARLES LEWIS POTTER, M. Am. Soc. C. E.*

Died August 6, 1928.

Charles Lewis Potter, the son of Benjamin R. and Susan E. (Smullen) Potter, was born at Lisbon Falls, Me., on January 24, 1864. His boyhood was passed on a farm in his native State and his early schooling was obtained in the public schools in the vicinity.

Mr. Potter entered the United States Military Academy at West Point, N. Y., in 1882, and was graduated therefrom in 1886, standing fifth in his Class. He was commissioned a Second Lieutenant and assigned to the Fifth Cavalry, on July 1, 1886. He served on frontier duty at Fort Supply, Indian Territory, in September and October, 1886, and then in garrison at Fort Leavenworth, Kansas, until March 29, 1887. He was transferred to the Corps of Engineers, on February 2, 1887, and was sent to the Engineer School of Application at Willets Point, N. Y., from which he was graduated in 1889.

Lieut. Potter's first Engineer assignment came in October, 1889, when he was appointed Assistant to the Engineer Officer in charge, at Montgomery, Ala., on the improvement of rivers in Alabama, Georgia, and Florida. His subsequent assignments were distributed over a great part of the United States as follows: Fortifications in San Francisco Harbor, and rivers in Southern California, 1891-1897; rivers in Oregon, 1897-1898; the Mississippi River at Memphis, Tenn., 1900-1903; the Great Lakes, Duluth, Minn., 1903-1906; Porto Rico, as Chief Engineer, Lighthouse Service, 1907-1910; the Mississippi River, at St. Louis, Mo., 1910-1912, and at St. Paul, Minn., 1912-1915; rivers and harbors at Portland, Ore., 1915-1916; at Boston, Mass., 1916-1917; at San Francisco, Calif., including fortifications, 1918-1920; and, finally, as President of the Mississippi River Commission, at St. Louis, from March 19, 1920, to June 12, 1928.

He was promoted to the rank of Captain on July 5, 1898; Major, September 14, 1904; Lieutenant-Colonel on February 27, 1911; and Colonel on November 27, 1916.

During the Spanish-American War and the Philippine Insurrection, Colonel Potter was Chief Engineer, in the Philippines, of the 8th Army Corps, on the Staffs of Generals Merritt and Otis (1898-1899), and during the World War he was Director of Gas Service (now the Chemical Warfare Service) at Washington, D. C. (1917-1918).

Colonel Potter's outstanding work was as President of the Mississippi River Commission. He was a man of very attractive personality, cordial and frank. He readily secured the confidence and respect of the leading men with whom the Commission had to deal, which had a far-reaching effect on harmonizing its many interests. During this service he was also Division Engineer of the Western Engineer Division, comprising river improvement works in the Engineer Districts in the Mississippi Valley from the head-waters to the Red River, including the rivers tributary to the territory of the Commission, except the Ohio and Illinois Rivers, and the Mississippi below Red River.

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* Memoir prepared by William S. Mitchell and James W. Skelly, Members, Am. Soc. C. E.

His greatest work came in 1927, following the phenomenal and disastrous flood of that spring in the Lower Mississippi Valley. During the months following this flood, to the end of 1927, he labored unceasingly in the preparation of comprehensive plans for the control of this great river, to be submitted for the consideration of Congress, frequently appearing before its Committees in advocacy of the Commission plan for the solution of the great problem. There is no doubt that the strain under which he labored so long and so assiduously weakened and undermined his health.

On reaching the military age for retirement, on January 24, 1928, he was retired, but was immediately recalled to active service, continuing as President of the Mississippi River Commission. The Flood Control Act of Congress, pending at the time, carried the provision that the President of the Mississippi River Commission should have the rank of Brigadier-General. Recognizing the importance of the Commission and of Colonel Potter's great service thereon, he was promoted to that rank on May 15, 1928.

Deeply interested in the vast work to be carried on by the Commission, he looked forward with eager anticipation to its prosecution. His wish was not fulfilled. To his great surprise and disappointment he was relieved from active duty on June 12, 1928, and shortly after was forced to undergo what was expected to be a comparatively safe surgical operation. His impaired physical condition, however, left him unable to rally from the shock of the operation, and he passed away quietly, on August 6, 1928.

General Potter made readily, and retained loyally, friends in every walk of life. He loved Nature and the outdoors. He was a keen student of archæological and historical matters and of fraternal subjects concerning which he possessed a fine library. He contributed numerous articles to engineering journals concerning the river problems before him. These were widely read and appreciated for sound engineering wisdom, great common sense, and clarity of statement.

He was a member of many Masonic bodies, having joined the organization in Memphis, where he was advanced to the Thirty-third and last Degree of the Ancient and Accepted Scottish Rite in a most unusually short time. In spite of frequent changes of station incident to Army life, General Potter was one of the few officers ever to attain this distinction. His funeral services were held in the Scottish Rite Cathedral at St. Louis, and his ashes were deposited in Valhalla Mausoleum near-by.

General Potter is survived by his widow, Mrs. Sophie H. Potter, and his step-daughter, Caralisa Nichols.

General Potter was elected a Member of the American Society of Civil Engineers on April 1, 1903.

HENRY GOSLEE PROUT, M. Am. Soc. C. E.*

Died January 26, 1927.

Henry Goslee Prout was born in Falls Village, Va., on August 10, 1845, the son of William and Amanda (Goslee) Prout, and a direct descendant of Timothy Prout who came from Biddeford, Devonshire, England, and landed in Boston, Mass., in 1644.

As the family had moved from their Southern home to New York and, later, to Massachusetts, Mr. Prout's boyhood was spent entirely in Northern surroundings. His education, prior to the Civil War, was not a very systematic one, but was gleaned intermittently at Stockbridge (Mass.) Academy and under the private tutelage of the Rev. Franklin Le Barron. In 1863, at the age of 18, he enlisted in Company D of the 57th Regiment of Massachusetts Volunteers, and served with the Army of the Potomac in the Wilderness Campaign, the siege of Petersburg, and the resulting pursuit of General Lee. In 1865, he was mustered out of service and after a two-year period of adventure and study, entered the University of Michigan, at Ann Arbor, Mich. He was graduated in 1871 with the degree of Civil Engineer, later receiving the honorary degrees of Master of Arts from Yale University in 1902 and Doctor of Laws from the University of Michigan in 1911.

During his second year in college Mr. Prout commanded an expedition of reconnaissance in Southwestern Colorado and so distinguished himself by his astronomical achievements that he was brought to the notice of Gen. William T. Sherman, then at the head of the U. S. Army and previously in command of the Military Division of the Missouri. In consequence, Mr. Prout was one of six young men who, at the request of the Khedive of Egypt, was recommended for military service by General Sherman. He was commissioned as Major of Engineers and remained in Egypt four and one-half years, during which time his experiences were both humorous and tragic, and his strength and endurance were given the most severe of physical and mental tests. In these years of service he rose to the position of Colonel in the General Staff and became Governor General of the Provinces of the Equator, relieving Gen. Charles George Gordon when he went to Khartoum as Governor General of the Sudan. An interesting problem which Colonel Prout undertook while he was in the Equator Provinces was the transportation of an 80-ft., screw-propelled steamer which was to be erected at Dufli, across 60 miles of broken country on the backs of negro bearers. It was in this steamer that Emin Pasha was sent to meet Stanley at the time of the famous "rescue".

On his return to America, Colonel Prout was employed as Signal Engineer by the Toucey and Buchanan Interlocking Switch Company, of Harrisburg, Pa., which later developed into the Union Switch and Signal Company. He remained in this position, however, for only a little more than a year, and in 1880 entered the printing business with the firm of Atkin and Prout, publishers of *The Railroad Gazette*. In March, 1887, he assumed the Editorship of this journal, which position he held with distinction for sixteen years.

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* Memoir prepared from information on file at the Headquarters of the Society.

At the expiration of this time, his interests were again directed to the Union Switch and Signal Company with which he had been associated and, in consequence, in 1903 he resigned his position as both Editor and Director of *The Railroad Gazette* to accept the office of First Vice-President and General Manager of the Union Switch and Signal Company, at Swissvile, Pa. He resigned this position in July, 1914, shortly after the death of the late George Westinghouse, M. Am. Soc. C. E., in whose general business interests he had been an adviser, and in November of that same year became President of the Hall Switch and Signal Company, of Garwood, N. J.

In 1915, Colonel Prout retired, giving up at this time all active business pursuits. It was during this period of his life, while he was living at his home in Summit, N. J., that he wrote the well-known biography of George Westinghouse, published in 1924, which, in addition to being an accurate portrayal of its subject, also sheds great light on the character and personality of its author.

In his diversified career, Colonel Prout was an outstanding figure in military, editorial, and professional achievement. His early associations with the great world figure, General Gordon, his startling editorial read on the floor of the United States Senate to which was due the choice of Panama instead of Nicaragua as the route for the Isthmian Canal, and his skill in mercantile leadership and its allied associations with the switch and signal companies—all lend unending tribute to his name. A distinguished scholar and a gentleman, he is well described in the following sketch of appreciation by his lifelong friend and associate, Mr. William H. Boardman:

> "The sum of it is that he is inspiring. He has such a fund of general and specific information, such a love of truth and thoroughness, such a scorn for lying and carelessness, such a ready quality of human sympathy that, with a faculty for memory cultivated and quickened by editorial habit, he can subordinate himself and skillfully direct conversation so as to draw out others."

Colonel Prout was a member of the American Geographical Society, the Sigma Phi Fraternity, the Century Club, the Railroad Club, and the University Club, of New York, N. Y.

He was married at Fort Leavenworth, Kans., in 1877, to Gabriella Perin. He is survived by his widow, two sons, Henry B. and Curtis, and three daughters, Elizabeth, Mrs. Pierpont V. Davis, and Mrs. Paul G. Tomlinson.

Colonel Prout was elected an Associate of the American Society of Civil Engineers on November 6, 1872, and a Member on September 3, 1879. He served as a Director of the Society from 1893 to 1895.

JOHN RICHARD RABLIN, M. Am. Soc. C. E.*

Died September 17, 1928.

John Richard Rablin was born in Plymouth, England, on October 25, 1868, the son of William H. and Mary Edwards Rablin. He came to the United States with his parents when he was about three years of age.

He was graduated from the Boston Latin School, Boston, Mass., and entered Harvard College, but the death of his mother, which caused the removal of the family to Maine, ended his college education.

Mr. Rablin began his technical career in 1888 as an Engineering Assistant on the Old Colony Railroad, under the late B. T. Wheeler, M. Am. Soc. C. E. His professional education was obtained under Mr. Wheeler's tutelage, and by studying nights by himself, while he was working during the day. When Mr. Wheeler went to the City of Boston as Superintendent of Streets in 1895, Mr. Rablin went with him, remaining there until some time in 1896, when he became associated with the contracting firm of Horton and Hemenway, as Contractor's Engineer. This firm had contracts in connection with the construction of the South Terminal and Back Bay Stations. The principal work on which Mr. Rablin was engaged was the construction of the Back Bay Station.

His service with the Metropolitan Park Commission commenced in 1899, when he was made Assistant Engineer in charge of construction of the Charles River Speedway. In 1904, he was appointed Chief Engineer of the Commission, a position which he retained until his death.

There was a tremendous expansion in the amount and kind of work done in the Metropolitan Park System after Mr. Rablin assumed the position of Chief Engineer. The work was not only construction and maintenance of parks, but also the construction of many important boulevards and their later improvement to enable them to carry heavy automobile traffic, as well as the construction of a number of bridges in the district. The great diversity of his work would be better understood, perhaps, if it were mentioned that he also had charge of shore protection and the construction and maintenance of sea-walls at various points along the shore in the general vicinity of Boston.

Among the works which Mr. Rablin constructed were the Revere Beach Parkway; a dam at Newton Lower Falls, Mass.; the Lynn Shore sea-wall; the Nantasket Beach sea-wall; shore protection at Revere and Winthrop, Mass.; dam, tide-gates, and boat lock at Medford, Mass.; the Northern Artery; and approximately twenty-eight bridges, the majority of which were spans of considerable size. Among these are the new bridge on the Old Colony Parkway, the Larz Anderson Bridge, over the Charles River, and the Cottage Farm Bridge.

He was continually reading and seeking knowledge regarding up-to-date methods and the latest developments in engineering along all the various lines into which the wide diversity of his work led him. He showed a great deal of ingenuity in varying types of construction to avoid duplication.

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* Memoir prepared by Lewis E. Moore, M. Am. Soc. C. E.

The following appreciation by Chairman Keniston, of the Metropolitan District Commission, was printed in *The Boston Transcript* shortly after Mr. Rablin's death:

> "An able and devoted servant of the Commonwealth, John R. Rablin, passed away this week, Mr. Rablin became Assistant Engineer to the Metropolitan District Commission in 1899, was appointed Chief Engineer in 1904, and continued in that capacity until his death. During that time the construction of many public improvements in metropolitan parkways, bridges, and reservations came under his direct care. The wise planning and satisfactory completion of these public works was largely due to his engineering skill and attention.
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> "Mr Rablin's life work was the development and maintenance of the Metropolitan Parks. He was ever ready to give freely of his time and strength, and his early death may be attributed in part to his attempt to resume too soon his duties after a serious illness. He still maintained an interest and oversight of his work as long as his strength permitted. The name of the engineer may not be known or remembered by those who enjoy the benefits of the public improvements for which he was responsible, but the works themselves will remain as a tribute to his earnest and successful efforts. May the public occasionally think gratefully of such men as Mr. Rablin."

Mr. Rablin was a man of very decided character, strong in his likes and dislikes, and much beloved by his friends. He was a member of the Society of American Military Engineers; the Boston Society of Civil Engineers; the American Road Builders' Association; the Massachusetts Highway Association; the International Road Congress; the Boston City Club; and the Wollaston Golf Club. He attained the ultimate in golf by "making a hole in one."

On November 15, 1893, Mr. Rablin was married to M. Frances Loring. He leaves, in addition to his widow, a son, Richard Loring Rablin.

Mr. Rablin was elected a Member of the American Society of Civil Engineers on August 31, 1909.

JOHN CHESTER RALSTON, M. Am. Soc. C. E.*

Died July 15, 1928.

John Chester Ralston was born in Kincardine, Ont., Canada, on May 12, 1864, his parents being James Glendenning Ralston, of Paisley, Scotland, and Mary Amanda (Johnston) Ralston, of Otterville, Ont., Canada.

His father was descended from an old Scottish family, the McDuffs, Thanes of Fife of Renfrewshire, the earliest records thereof being dated 1190. The last son of that ancient family, Ralph, on receiving a grant of land, named a part of it for himself, Ralph's Town, afterward abbreviated into Ralston—hence, the family surname.

Both Mr. Ralston's father and grandfather were graduates of the University of Edinburgh. His mother was a descendant of the Johnston and Harris families of Massachusetts and Rhode Island, noted in the history of the Minute Men of the Revolution. One of her earlier forebears is credited with being the author of the first book ever written in America, a quaint volume entitled "God's Dealings with Zion's People".

Mr. Ralston received his early education in one of the public schools of Ontario, under a Head Master who afterward became Minister of Education for the Dominion of Canada. There, the foundation was laid for a liberal academic course to be developed in either a Canadian or an American university; but these plans were suddenly terminated by the death of his father, who left him, at the age of thirteen, the eldest of nine children. Thereupon the inspiration of the classroom had to be replaced by the harsher curriculum of self-support. Having seen the metes and bounds of his father's property re-surveyed, young Ralston developed a nebulous hope of becoming a surveyor and ultimately an engineer.

In 1880, he went into the West and found early employment in the United States Engineer Corps, under the direct supervision of Mr. Charles S. Pease, on the reclamation of the Missouri River, at the beginning of the Federal improvements on the Missouri, Mississippi, Ohio, and their tributaries. It was under the encouragement and personal help of Mr. Pease and his Staff that Mr. Ralston's engineering self-education was directed and inspired. The members of the Staff were largely graduates of the Rensselaer Polytechnic Institute, who thus exemplified in their lives a generous spirit of service.

Under this tutelage the curriculum of the Rensselaer Polytechnic Institute was followed by difficult, arduous, and, at times, most discouraging effort. The nature of the pioneering and improvement plans of the Missouri River, however, was ever fascinating and inspiring, and the discouragement of lamplight study was always transmuted during the day into Aladdin's mysteries. The wizardry of the laws of hydraulics, the nebulous latitudes of river regions and channel erosions, the numberless problems, and the never-ending devices for control or remedy, were a challenge to engineering study. Under such special environment the college classroom as an educating medium was surpassed.

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* Memoir prepared by J. A. L. Waddell, M. Am. Soc. C. E.

In later life Mr. Ralston undertook a broadening, non-technical course of study, following the curricula of Columbia University and the University of Chicago.

In 1882 he was transferred from the Missouri River at Omaha, Nebr., to the Potomac River, on the reclamation of the Potomac Flats at Washington, D. C., and Georgetown, Md., Harbors, and on the building of what afterward became Potomac Park, Washington, D. C. He was given responsible charge of the construction of the mattress foundation of the sea-wall behind which the channel dredgings were deposited to form the terra firma of this interesting park. This sea-wall was more than five miles long, and required several seasons to complete. The wall defined the final north shore line of the main channel of the Potomac River from Georgetown Harbor, past the historic Long Bridge, to a confluence with the south shore of the Washington Channel entrance to the city docks, enclosing a peninsular area that separates the two main navigable channels. This area reclaimed a large and offensive part of the mud flats of the Potomac River, while the dredgings from the widened outside channels supplied the necessary material to raise the flats well above high tide and to create a fertile acreage for landscape gardeners. More than 400 acres were thus reclaimed.

When, in 1885, the major part of the reclamation had been fairly completed, Mr. Ralston was tendered a position as Assistant Engineer on the Construction Staff of the Union Pacific Railroad. For the first two years of his railroad experience he was employed directly by the late Jacob Blickensderfer, M. Am. Soc. C. E., Chief Engineer, principally in collaborating the history of the Pacific Railroads, on the building of which Mr. Blickensderfer, together with the late Grenville M. Dodge, Hon. M. Am. Soc. C. E., and the late Adna Anderson, M. Am. Soc. C. E., had served so conspicuously.

Minor executive duties also were required of young Ralston in the line of reports, and in furnishing supplies for the many field parties of location and construction in the several Western and Northwestern States crossed by this road. Most valuable experiences were gained and studies pursued under the kindly and inspiring direction of this new Chief, who always took the liveliest personal interest in the welfare and education of his assistants. Three nights a week were devoted to study and instruction in the astronomical observatory of Mr. Blickensderfer, whose work in this field was well and favorably recognized by the United States Astronomical Observatory at Washington. Here the principles of railroad engineering and the early basis of railroad economics, in both engineering and operation, were studied with a dogged determination and with a lively appreciation, in the realization that a master of this temper and vision was not to be found every day.

In 1887, a new Chief Engineer was appointed in the person of the late Virgil G. Bogue, M. Am. Soc. C. E., who early granted Mr. Ralston his long cherished wish to be put into the field. An extended experience in preliminary and final location in many of the rugged Western States, including Colorado, Wyoming, Nevada, Western Montana, Eastern Washington, and Idaho, and on construction work in Kansas, Nebraska, Colorado, Wyoming, and Montana, gave a diversity and comprehension of the widest application of railroad engineering, such as seldom come to young men. Mr. Ralston was given charge of the remodeling of the Butte City, Mont., yards to meet the growth of the expanding mining industries of that region. In that work he was called on to examine and report on the first water-power sites contiguous to the smelter region that later was to be so remarkably developed.

Other notable work arising out of that time and region was the preliminary and final location of the high-line development of what, in later years, became the famous Butte, Anaconda, and Pacific Railway, the first electrically-operated, heavy-freight carrying railroad in the world. To these works and allied activities Mr. Ralston, as Assistant Engineer, contributed with distinction.

In 1889 and 1890, he took charge of the preliminary surveys and much of the final location of the west end of the Pacific Short Line through Wyoming and Utah, employing and directing a half dozen field parties. He succeeded in shortening the main-line distance 90 miles between the Missouri River and Ogden, Utah, and in finding a most economical crossing of the Continental Divide, nearly 1 200 ft. lower than that of the near-by competitor. Across the uplands of the Big Sandy drainage he was able to locate a long tangent on a 0.5% grade, at exceptionally low cost, the east end of which was within eight miles of the Continental Divide. In 1890 the first period of his career ended, when a suspension of active railroad development in the West was declared.

Mr. Ralston immediately took up steel skeleton building construction, foundation work, and elevated railway erection in Chicago, Ill., and pursued this class of occupation continuously until 1897. During this period he erected a number of early steel skeleton buildings and special structures, especially for the Illinois Steel Company in South Chicago, and for the Lake Street Elevated Road, including the down-town loop along Lake Street to and down Wabash Avenue to Harrison Street, etc. This erection constituted the first down-town loop, and was quickly followed by the joint loop structures, located in what has since developed into the most congested metropolitan district of the world.

Several notable steel skeleton structures were erected in Buffalo, N. Y., Pittsburgh, and Philadelphia, Pa., including the Ellicot Square Building, in Buffalo, and the Presbyterian Building, in Philadelphia, each notable in its day for size and construction.

In the midst of these activities, in the spring of 1897, Mr. Ralston was called to Spokane, Wash., by some of his earlier friends of Butte, Mont., to assist in certain mineral developments and in the construction of reduction works. He assisted in the design and construction of the largest plant of its kind in the State of Washington; and in this, the third period of his career, he established himself as a Consulting Engineer, his practice quickly growing into substantial proportions. It covered examinations, reports, and valuations in the fields of hydraulic development, hydraulic power, and later, hydro-electric power in British Columbia, Washington, Oregon, Idaho, California, Montana, and Nevada, including a fascinating diversity of technology.

Intense activities in those fields, especially in California, led to an examination of the water supply and power possibilities of the Tuolumne and its Hetch Hetchy Valley, the Stanislaus, and the Mokelumne, out of part of which, and certain parts of the Merced, was finally carved the so-called Hetch Hetchy water supply for San Francisco, Calif. Another interesting work was his study of the hydraulics involved in the disposal of the underground waters of the famous Comstock Lode, their siphonic behavior, vadose waters, accumulations, and colonization. A year was spent on this work and on the plans for the complete unwatering of certain of the leaner parts of the Lode, involving a blending of the technology of the Civil Engineer and the rudiments of Economic Geology.

In April, 1906, the San Francisco earthquake destroyed much of that beautiful city and caused Mr. Ralston to be sent to the hospital, where a major operation became imperative, and rendered him a permanent invalid. This put an end to field activities for all time, and left to him as a residue only a limited consulting practice. Notwithstanding these almost insuperable physical handicaps, he was able to take up the duty of re-organizing the Engineering Staff of the City of Spokane, Wash., which was tendered him at that time. He succeeded in bringing into that sadly depleted organization an *esprit de corps* and a technological staff of singular merit, one comparable to the best metropolitan standards. Under the atrophying influences of indurated city politics, a brief but decisive encounter followed, resulting in the establishment of a permanent and high-minded Engineering Corps. Under this régime Mr. Ralston was able to replace eight old and dilapidated wooden Howe truss bridges by designing and building an equal number of reinforced concrete structures, literally making Spokane the "City of Bridges". Among these structures is the famous Monroe Street Bridge, said by many to be the most monumental concrete arch bridge in the United States. The main deck is a segmental arch of 290 ft. span, with a rise of 140 ft. Several of the other bridges are notable in design, and are representative of the true arch bridge.

Large paving areas were inaugurated for the first time for the city; and a new underground water supply of exceptional quantity and purity was designed and built under Mr. Ralston's direction; thus the struggling Western outpost was transformed into a modern metropolitan city. This city-building activity demanded a liberal and wide technology, with a generous spirit of service on the part of the entire staff.

Mr. Ralston was ever an earnest advocate of the principle that the engineer always should take the leadership in professional, civic, and political life. He maintained that the engineer by native mental qualities and education is more fitted for leadership in the activities of men and society than the lawyer, the divine, or the physician. He especially insisted that the engineer should have a command of forceful, fluent, and happy diction commensurate with the leadership to which he should aspire. He lived to see the day when the best thought in the profession is becoming vocalized and insistent on this subject, in direct contrast to two or three decades ago when the engineer was seldom an advocate on any subject except engineering.

In respect to the use of the English language, Mr. Ralston practiced what he preached; for both his speech and his writings were conspicuous for an extensive vocabulary and a fluent diction. Occasionally, some of his brother engineers smiled over his use of long or unusual words, but those words were always well chosen, apropos, and forceful.

During his invalid years from 1906 until his death, aside from his work as City Engineer, he found enough physical ability to report on a number of hydro-electric projects and on straight hydraulic developments aggregating 750 000 h.p., in units varying from 3 000 to 50 000. Prior to the entrance of the United States into and during the World War, Mr. Ralston served as a member of the Naval Consulting Board for the State of Washington—a period extending from 1915 to 1918. During this time he was also one of about 2 000 entrants who were asked to submit plans and specifications for the dissemination of American propaganda among the German people. His plans were accepted; and he was one of the first four to receive Presidential recognition and a monetary reward.

He also served as a member of the Committee on Development of the Society, sharing actively in its deliberations during 1918-1920. From 1920 to 1924, Mr. Ralston served as a member of Engineering Council. In this Council he represented, as the Executive Member, the States of California, Nevada, Oregon, Idaho, Washington, and Montana.

From 1900 until 1918, he acted as Consulting Engineer, and, for a part of that time, as Vice-President, of the Pacific Coast Pipe Company with a factory at Seattle. This Company manufactured all sizes of wood-stave pipe, notably the continuous-stave type, up to diameters of 120 in. Its use was mostly for penstocks and pressure pipe for hydro-electric plants and for inverted siphons in irrigation. He was one of several engineers of the Pacific Coast who developed and made popular this pipe for more extended use, and this work properly may be said to have been a great contribution to the current engineering development of the time.

He was an early worker in the Boy Scout movement, a pioneer in social hygiene work, a deep lover of the arts, and ever willing to aid in the betterment and beautifying of his city, having planned outlines for the economic and beauteous development of the falls and river of Spokane. He was President of the Art Association, and a brilliant, original, and much sought speaker. He never feared to tell the truth, possessing a decided vision of life and of engineering, which also found expression in his great enjoyment in writing. He was interested in and always ready to assist young men.

While Mr. Ralston was certainly a self-made man, he differed essentially from most of the American engineers of this class. Not only was he a great reader and student, but his studies were laid out systematically and were followed religiously, so that eventually he was in no whit inferior to the leading alumni of the best technical schools.

For two decades the writer had held Mr. Ralston in the highest esteem, having greatly admired him as a man, a citizen, and an engineer—as, truth to tell, did every one else with whom he was thrown into intimate contact.

In corroboration of this statement, quotations from a few of the letters of sympathy received by Mrs. Ralston will serve.

That from an associate reads:

> "He was a fine citizen, a capable and resourceful engineer; and as a friend I have not known his superior."

That from a subordinate is:

> "The great pleasure of working for and with 'Ralston' was the being in contact with a man of vision; for to him engineering was not the mere application of formula and instrument, but combined with these the foresight to conceive projects which should lead men's feet over the hills to an era."

Of the Humboldt County, California, Project, it was stated:

> "We undertook a very daring project; and, thanks to his skill, accomplished it with brilliant success. The North Mountain Power Project was one of the first of its kind in the West, and its record for time has never been beaten."

M. E. Cooley, M. Am. Soc. C. E., wrote:

> "My acquaintance with Mr. Ralston dates from his service on the Board of Federated Engineering Societies. I learned then of his sterling qualities, his high ideals, and the inspiration of his presence."

That from L. W. Wallace, President, Federated Engineering Societies, read:

> "He lived a life of rich accomplishment and endearment, a rich heritage to his friends. He will be mourned and missed in the Engineering Profession, to which he contributed a great deal."

A College President stated:

> "His understanding of his associates was an indication of his very generous and universal kindliness."

A Bishop of the Protestant Episcopal Church wrote:

> "He was a rare man. He was a true and loyal man in all the relations of life. His ability, courtesy, and consideration for others always impressed me greatly."

Mr. Ralston's day is past; his work is done; but it will be many, many years before he is forgotten. The history of such a life should be an inspiration to young men who desire to become engineers, but who are unable to secure money for a regular course of technical instruction.

Mr. Ralston was married in New York, N. Y., in April, 1897, to Mary Kean Buckner, of Kentucky, whose family came from England to Jamestown, Va., in Colonial days. He is survived by his widow, a son, John W. B., a daughter, Mary Elizabeth, two brothers, and five sisters.

Mr. Ralston was a member of the Spokane Engineering Society, the Seattle Engineering Society, the American Institute of Mining and Metallurgical Engineers, the Cosmos Club of Washington, D. C., the Spokane City Club, and the Sons of the American Revolution. He was also a Thirty-second Degree Mason, a member of the Protestant Episcopal Church, and a staunch Republican.

Mr. Ralston was elected a Member of the American Society of Civil Engineers on October 3, 1906.

BEVERLEY STROTHER RANDOLPH, M. Am. Soc. C. E.*

DIED FEBRUARY 5, 1929.

Beverley Strother Randolph was born on July 17, 1851, at Berkeley Springs, Va. (now West Virginia). He was the eldest son of the late James Lingan Randolph, M. Am. Soc. C. E., Chief Engineer, and, later, Consulting Engineer, for the Baltimore and Ohio Railroad Company, and the late Emily (Strother) Randolph, a sister of Gen. David Hunter Strother.

Mr. Randolph attended Cornell University for two years and then went into active civil engineering work. He was engaged for some time on location for the Baltimore and Ohio Southwestern Railroad Company (now a part of the Baltimore and Ohio System). He then went to Nicaragua on the construction of the first railroad to cross that country. On his return to the United States, he was appointed Superintendent of the mines of the Consolidation Coal Company, with headquarters at Frostburg, Md., which position he held for twenty years. During this time it is interesting to note that not a single strike occurred among his men.

In 1904, Mr. Randolph retired from active engineering work to his farm at Berkeley Springs, W. Va., where he bred horses for the New York market. He later moved to "Tonoloway" near Hancock, Md., where he continued raising horses and also operated a glass sand deposit on his farm. In 1927, at the age of 76, he sold "Tonoloway" and bought a farm, "Hawkhurst", in Berkeley County, West Virginia, near which he was killed in a grade crossing accident on February 5, 1929.

Mr. Randolph was a man of remarkable physique and at the age of 72 was still schooling horses to jump. At the time of his death, he was making plans to go into the breeding and training of trotting horses on a more extensive scale than he ever had attempted before.

He was a Member of the Order of the Cincinnati, and a Life Member of the American Institute of Mining and Metallurgical Engineers.

On September 20, 1882, he was married to Mary Jewette, of Baltimore, Md., who died on December 26, 1918. He is survived by his three nephews, James Robbins Randolph, O. Robbins Randolph, and Lingan Strother Randolph, Jr., and one niece, Mrs. Emily Randolph Deitrick. His brother, the late Lingan Strother Randolph, M. Am. Soc. C. E., died in 1922.

Mr. Randolph was elected a Member of the American Society of Civil Engineers on May 2, 1888.

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* Memoir prepared by O. Robbins Randolph, Esq., Rutherford, N. J.

HARRY FAY ROACH, M. Am. Soc. C. E.*

DIED JULY 28, 1929.

Harry Fay Roach was born in St. Louis, Mo., on May 7, 1871, the son of Harry E. and Sarah (Haley) Roach. His mother was a native of Kalamazoo, Mich. His father was an Architect and Builder and the son grew up in an atmosphere of construction, spending his spare time in his father's office and on work in the field. He thus acquired considerable valuable training in his early youth.

Mr. Roach was graduated from the St. Louis Manual Training School in 1889, and in the fall of that year entered the Massachusetts Institute of Technology. He remained there until the summer of 1891, taking the course in Architecture, after which he entered into partnership with his father for the practice of his profession in St. Louis. In 1894, he traveled through England, France, Belgium, Germany, Switzerland, Italy, and Greece, studying the architecture of those countries. His partnership with his father was terminated in 1903, and thereafter, until the time of his death, he was engaged in architectural and engineering work. He designed many important buildings in St. Louis, among which are the Hamilton-Brown Building, the Times Building, the Buckingham Club, and the Syndicate Trust Building.

Mr. Roach was a mathematician of note and had an ingenious mind and mechanical ability of high order. He developed the means of generating electricity for the lighting of railway trains as early as 1899. He patented railway joints and angle-bars, designed rail sections, and invented a method of photographing the deflections in railway track and structures under moving loads by the use of motion picture camera and reflectors. All the work of invention and research that Mr. Roach did, was actuated by the love of scientific investigation and not with the hope of any pecuniary reward. His discussion of engineering subjects was always instructive, and his friendship was prized by a large circle of professional men in St. Louis.

Mr. Roach was affiliated with the following organizations: The Third Baptist Church, of St. Louis; the Masonic Order; the American Institute of Architects; the Society of American Military Engineers; and the St. Louis Railway Club. After a severe illness of three months, he passed away on July 28, 1929.

In 1893, he was married to Mary E. Gallop, who survives him. He leaves also two daughters, Mrs. Donald B. Pheley, of Pasadena, Calif., and Dorothy E. Roach, of St. Louis; and four sons, Harry F. Roach, Jr., and Alan Douglas Roach, of St. Louis, Alden G. Roach, Assoc. M. Am. Soc. C. E., of Pasadena, Calif., and Noyes H. Roach, of Joliet, Ill.

Mr. Roach was elected a Member of the American Society of Civil Engineers on January 19, 1925.

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*Memoir prepared by a Committee of the St. Louis Section consisting of F. G. Jonah, C. D. Purdon, and Baxter L. Brown, Members, Am. Soc. C. E.

THEODORE CHARLES ROBERTS, M. Am. Soc. C. E.*

DIED SEPTEMBER 28, 1928.

Theodore Charles Roberts was born in New Orleans, La., on December 3, 1875. He spent his boyhood there and, simultaneously with his early school days, worked in a system of shops owned by his father, thus acquiring a knowledge of mechanics and manufacturing, together with his elementary education. Following his public school preparation, his education was continued at Tulane University, at New Orleans, and at Colorado College, Colorado Springs, Colo. He was a tireless student both through his school and college days and throughout his life. Were it a problem in mathematics or mechanics, a question of industrial or political economy, the design of an industrial plant, the location of a railroad, the management of a high enterprise, the purchase of equipment, or a game of bridge, his ever active mind was alert and eager.

His dissipation, like that of many other engineers and authors, was in his work. His rise in the industrial world was rapid. Always eager for mental advancement, he spent approximately six years teaching mathematics while serving as Field Engineer and Construction Superintendent for the Canon City Light, Heat, and Power Company, as Manager for several combined exchanges for the Colorado Bell Telephone Company, and as Construction Superintendent and Engineer for the Telluride Power Company, Telluride, Colo. At this time, while he was associated with Mr. Ralph D. Mershon, a number of high-tension electrical principles were established.

Subsequent to these engagements, Mr. Roberts became Manager of several mining and milling companies in Colorado. With the experience accumulated up to this time he entered the field of Consulting Engineering and Contracting, at Leadville, Colo. His keen insight, strong personality, and executive ability created a demand for his services with the United States Reduction and Refining Company as Engineer in charge of all design, construction, and operation for its various plants.

From 1904 Mr. Roberts served two years as Chief Electrical Engineer for the American Smelting and Refining Company, in charge of all electrical equipment. His efficiency here caused his transfer to the position of Chief Engineer and Master Mechanic in charge of engineering and mechanical operations for the Pueblo, Colo., Plant.

In 1908 the electrical industry had exceeded all expectations and left the works inadequate for the demand. This exigency drafted Mr. Roberts into the position of Chief Engineer and General Superintendent for the Arkansas Valley Railway, Light, and Power Company, at Pueblo, Colo., a property of H. M. Byllesby Company, of Chicago, in charge of the rebuilding, expanding, and bringing up to date its steam and hydro-electric power plants, street railways, high and low-tension transmission lines, and the electrification of various industries to be served by them.

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* Memoir prepared by W. R. Palmer, M. Am. Soc. C. E., and John G. L. Cunningham, Esq., New York, N. Y.

By 1912 the expanding copper industry in Arizona had outgrown entirely its reduction plants which caused a complete re-organization of operations. The United Verde Copper Company, one of the properties of the late Senator Clark, as did the others, decided on this expansion and engaged Mr. Roberts for the work. It consisted of a new reduction plant, for 9 000 000 lb. of copper per month; a housing development for 10 000 people, with the attendant roads, streets, public utilities, steam and electric railroads, and land reclamation; as well as the management of the operations for the existing plant. In this position his wide experience and ability brought him into all the large engineering developments of the State.

With the beginning of hostilities in 1917, and prompted by his patriotic spirit, Mr. Roberts offered his services to the United States Government. He was commissioned a Major in the United States Army and became engaged in the manufacture of dyestuffs and chemicals.

Soon after the Armistice was signed he became associated with the phonograph industry, as Assistant to the President in Charge of Engineering and Manufacturing for the Columbia Graphophone Manufacturing Company, where, under his direction, the production of the plant was increased from \$15 000 000 to \$100 000 000 per year, gross sales. Unfortunately, the heavy responsibility and concentrated application for many years had so broken his health that thereafter he had to limit his services as a consultant to a few selected clients. His liberal disposition and interest in others caused him to devote himself through his spare moments to scientific and other writings in order to perpetuate as far as possible his broad experience.

Mr. Roberts died as simply as he had lived. He attended his club in the afternoon, passed the evening with his family, and joined the staff of his Creator very quietly before dawn on September 28, 1928.

In the world of recreation Mr. Roberts' intensive work was broken now and then by a skillful game of golf, a friendly bout with the gloves, a wild-game hunting trip, a cruise in his yacht, or a clever game of bridge. He was a member of the American Society of Mechanical Engineers, the American Institute of Mining and Metallurgical Engineers, the American Institute of Electrical Engineers, the National Electric Light Association, the American Institute of Architects, the Engineers and Columbia Yacht Clubs of New York, the Country Club of Norwalk, Conn., and the Country Club of Prescott, Ariz. He affiliated himself with the Masonic Fraternity early in life, working through the various degrees to that of Knight Templar and Master of the Royal Secret of the 32d Degree, and became a Noble of the Mystic Shrine.

Mr. Roberts is survived by his widow, Marion Wall Roberts, M. D., and a daughter, Margaret Mary. A son, Theodore Merton, died in 1916.

The sudden demise of Mr. Roberts, in the prime of life, took from the engineering world a thinker and a leader, and from his family, his friends, and his associates a boon companion, a genial host, and a gallant gentleman. Once more the Builder has called. In the ranks of the valorous another engineer of great distinction stands, in silent, smiling dignity—enobled by his service and sacrifice for humanity and enshrined with his peers in the perpetual glory of great accomplishment.

Mr. Roberts was elected a Member of the American Society of Civil Engineers on September 11, 1917.

ALBERT FOWLER ROBINSON, M. Am. Soc. C. E.*

DIED JANUARY 20, 1929.

Albert Fowler Robinson, the son of William Dirgin and Ruth (Fowler) Robinson, was born on October 12, 1853, of Puritan stock, at Henry, Ill.

He was graduated from the University of Illinois in 1880 with the degree of Bachelor of Science in Civil and Mining Engineering, and, in 1881, took a Post-Graduate Course in Bridge Engineering. In 1891, the University recognized his high scholastic attainments and fine record in his profession by conferring upon him the degree of Civil Engineer.

Immediately after his graduation and prior to taking his Post-Graduate Course, Mr. Robinson entered the service of the Denver and Rio Grande Railway Company as Chainman. In 1882, he became Assistant Bridge Engineer on the Chicago and Alton Railway, and, in 1883, accepted a position with the Union Bridge Company, Athens, Pa., as Calculator. His first service with the Atchison, Topeka, and Santa Fé Railway Company began in July, 1884, at which time he was appointed Bridge Engineer. In September, 1885, he accepted a similar position with the Chicago, Burlington, and Northern Railroad Company.

In 1889, Mr. Robinson became associated with the late E. L. Corthell, Past-President, Am. Soc. C. E., in Chicago, Ill., as Chief Designer; and, in 1891, he accepted a position as Chief Draftsman with Purdy and Henderson, Architects, in Chicago. In June, 1892, he became Assistant Engineer with the Chicago, Rock Island, and Pacific Railway Company, with which he remained until August, 1894, when he entered private practice. In 1896, he again entered the service of the Atchison, Topeka, and Santa Fé Railway Company as Bridge Engineer, with headquarters at Topeka, Kans. He was appointed Bridge Engineer of the entire System in 1901, with headquarters in Chicago subsequent to 1904, which position he retained until his death.

Mr. Robinson was recognized as an outstanding leader in his profession, and took an active interest in the work of the various engineering societies of which he was a member. He was a pioneer in many respects, notably in ballasted floor construction for railroad bridges and in timber preservation, as applied to railroad cross-ties and piling in particular. He looked upon his engineering work as more than a means of livelihood. In fact, he felt that it was his service to mankind to do his work well; and in truth he might, for upon the design of bridges depends the lives of many railroad passengers. These structures are enduring monuments to his work and stand in great numbers from Chicago westward to the Pacific and southward to the Gulf Coast. Two of the outstanding structures are the Missouri River Bridge, at Sibley, Mo., and the Mississippi River Bridge, at Fort Madison, Iowa.

In his various activities, Mr. Robinson made a host of friends, and to those who knew him intimately, his engineering achievements are overshadowed by his fine character. His keenness of mind, his exceptional ability, and his personal qualities endeared him to all, particularly to the younger men who frequently sought him for advice and guidance, and to whom he gave freely of his time and thought.

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* Memoir prepared by a Committee of the Illinois Section consisting of Messrs. Albert F. Reichmann, *Chairman,* and G. W. Harris, Members, Am. Soc. C. E., and J. de N. Macomb, Assoc. M. Am. Soc. C. E.

It was in 1885 that Mr. Robinson was married to Loretta Kate Elder, at Topeka, by whom, with three sons, Raymond Elder, Albert William, and Donald Bruce, he is survived. A fourth son, Phillip Edward, died in 1891. Mr. Robinson's family life was ideal.

He was a member of the American Railway Engineering Association, the Western Society of Engineers, the American Society for Testing Materials, the Chicago Engineers' Club, and Tau Beta Pi Fraternity. He was also a member of the Masonic Fraternity, the Glen Oak Golf Club, and the First Congregational Church of Oak Park, Ill.

Mr. Robinson was elected a Member of the American Society of Civil Engineers on November 2, 1887.

ARTHUR WELLS ROBINSON, M. Am. Soc. C. E.*

Died May 23, 1929.

Arthur Wells Robinson was born at Collingwood, Ont., Canada, on March 28, 1861. He was the son of Joseph Orr Robinson, a Solicitor of the Middle Temple, London, England, who at that time was engaged in legal work in connection with the town site of Collingwood, and of Georgiana Buchanan, the daughter of James Buchanan, H. M. Consul at New York, N. Y.

Mr. Robinson was educated at Leamington, England, and at St. Catharines, Ont., Canada, and the Brantford (Ont.), Collegiate. As a mere boy he showed great ability in mechanical invention and design and at the age of seventeen went to Montreal, Que., Canada, to work under the late John Kennedy, M. Am. Soc. C. E. (afterward Sir John Kennedy), Chief Engineer of the Montreal Harbor and St. Lawrence River Ship Channel. He remained with Mr. Kennedy for eight years during which time he was engaged on the design and maintenance of a fleet of forty dredges and vessels used in connection with the Ship Channel, and thus he obtained an intimate knowledge of this great waterway.

In 1885, Mr. Robinson opened an office in Philadelphia, Pa., as Consulting Engineer, but very soon became associated with the Bucyrus Company, of South Milwaukee, Wis., as Designer and, later, as Managing Engineer. In this capacity he laid the foundation of much of the standard practice in dredge and steam-shovel design and took out many patents. He also developed the American steam shovel, including the large types used in the iron ore mines of the Lake Superior Region. In 1900, he returned to Canada and practiced as a Consulting Engineer in Montreal and, later, was engaged by the Department of Public Works of Canada. In addition, he acted as Consulting Engineer to the Egyptian Government and the Government of the Sudan, as well as to the Crown Agents, for various Colonies.

At the request of the Egyptian Government in 1907, Mr. Robinson visited the Upper White Nile and made a report on the great swamp area, known as the Sudd Region. In this area much of the water of the White Nile was lost through evaporation, and the problem was to prevent the spreading of the flow from the main channel, thereby checking evaporation and saving the water for use in Egypt, 2 500 miles further down the river. In the Sudd Region the great swamp grasses and papyrus reeds grow 15 ft. high and cover the country for miles. Aided by his sound judgment and knowledge of the rank growth of this region, Mr. Robinson designed three dredges specially adapted to such work and by their use the river was confined to its proper channel and the flow very greatly increased, thus enabling many more acres to be irrigated in Egypt. He also designed a shallow draft self-propelling dredge to cut through the constantly shifting sands of the River Niger. This dredge was used in Mesopotamia during the World War—had it been sent there earlier, it would have aided greatly the Relief Expedition of the River Tigris.

During this period in Mr. Robinson's career he designed dredges for land reclamation work in many places, notably Lincoln Park, Chicago, Ill., which is land largely reclaimed from Lake Michigan. The same kind of work was done in Bombay, India, where large areas were reclaimed and added to the city by one of his machines. Many dredges, large and small, were also built from special and individual designs for harbor and river works in India, China, Japan, Siam, Burmah, and many other places. The Panama Canal was largely excavated by machines of his earlier type, and the Cape Cod Canal by two dredges specially designed for the purpose. In all, more than 400 separate and distinct dredges were constructed from his designs and this without counting several thousand steam shovels of which many, of course, are built of a single pattern. Scarcely a river or harbor in the world but shows his influence, directly or indirectly, and it is not too much to say that no man has made so great a mark on the physical surface of the world as Mr. Robinson. His active professional life lasted just fifty years.

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* Memoir prepared by F. E. M. Robinson, Esq., Upper Melbourne, Que., Canada.

He had a unique gift amounting to genius for mechanical design and this with habits of great industry enabled him to produce a constant stream of new and original designs and inventions until the very week of his death. Many patents were taken out in England and the United States and a still greater number of inventions, equally as original, were left unprotected. He utilized but little mathematics and used graphical methods to check his almost unerring instinct for appropriate size and strength of material. Many can bear witness to his almost uncanny facility in design, and to the speed with which he could produce a finished drawing of the new thoughts and inventions as they came to him. His was the creative power of a great artist working in steel.

For the last twenty years most of his dredges for use outside Canada and the United States were built by Messrs. Lobnitz and Company, Renfrew, Scotland, and with this firm he had the most pleasant relations. At the time of his death the Company was building several dredges from his design for use in the Sudan. Mr. Robinson returned early in April, 1929, from a five months' trip to Manchuria, Korea, and Japan where he inspected twenty-six harbors, and until just before his death was engaged in designing machines for work in these countries.

He was very devoted to the simple life and loved the quiet of the country; he was exceedingly fond of music and flowers and had a wonderful gift for landscape gardening; but greater than his achievements was the man himself: A devout Christian, Mr. Robinson lived in the realization of his Master—"seeing Him who is invisible." His faith was unshakable, his honor stainless, and, of him, his sorrowing friends may well say: "Whatever record leap to light, he never shall be shamed."

He was a member of the American Society of Mechanical Engineers, the Institution of Civil Engineers, London, England, and the Engineering Institute of Canada.

In 1892, he was married to Margaret Beatrice, daughter of the late T. M. Taylor, of Montreal, Que., Canada, who survives him, together with a son, F. E. M. Robinson, of Upper Melbourne, Que.

Mr. Robinson was elected a Member of the American Society of Civil Engineers on February 3, 1892.

EDWARD STANLEY SAFFORD, M. Am. Soc. C. E.*

DIED DECEMBER 31, 1929.

Edward Stanley Safford was born at Boston, Mass., on September 6, 1847. Following his graduation from the English High School in Boston in 1865, he entered, in the same year, the Massachusetts Institute of Technology which school had just been organized. He was enrolled as a member of the first graduating class, that of 1868, and took the course in Mining and Civil Engineering.

Mr. Safford was employed as Rodman on the first survey for the Atchison, Topeka, and Santa Fé Railroad, on July 7, 1868, from Topeka to Burlingame, Kans., a distance of 29 miles. Later, he was engaged on the long preliminary surveys, and, as Division Engineer, had charge of the construction of 45 miles of the road from Emporia to Florence, Kans.

In the fall of 1871, Mr. Safford laid out what is now the flourishing city of Hutchinson, Kans., on the Arkansas River, at the mouth of Cow Creek. He returned to Boston in the spring of 1872, and was engaged as Contractor on public works in and about Boston until 1878. He was Chief Engineer of the New London Northern Railroad Company from 1878 to 1880, after which he served as Division Engineer on the West Shore Railroad (then the New York, West Shore, and Buffalo Railroad) first at Newburgh, N. Y., in charge of construction in Orange County, New York, and, later, at the Buffalo Terminal. In 1884 and 1885, he was employed on the Philadelphia Branch of the Baltimore and Ohio Railroad, with headquarters at Upper Falls, Md. Subsequently, he established a home in Arlington, Mass. In 1885, he became Principal Assistant Engineer on the Mobile and Birmingham Railroad, remaining in Alabama from 1886 to 1888; later, he acted as Chief Engineer for the Shelby Iron Company, at Shelby, Ala.

In the winter of 1890-1891, Mr. Safford made a survey on snowshoes in Nova Scotia. From 1892 to 1895, he was Engineer in charge of construction of the Palisades Tunnel and the building of the two large coal-shipping trestles on the Hudson River, in addition to a terminal for the New York, Susequehanna, and Western Railroad Company, all of which work passed into the hands of the Erie Railroad Company soon after its completion.

In 1897, he went to Mexico and made surveys for the Chihuahua and Pacific Railroad Company from Chihuahua, Mexico, extending west for 200 km. He subsequently was chosen as Chief Engineer of this line, made location surveys, and finished the construction by May, 1900. Mr. Safford remained in Mexico until June, 1904, when he returned to the United States. He lived at Somerville, N. J., and at Sharon, Mass., for about three years, and then became Engineer for the contractors who were eliminating grade crossings on the New York, New Haven and Hartford Railroad, at New Bedford, Worcester, and Dorchester, Mass. In 1911, he went to Haiti on railroad construction. He returned to the United States the following year, thus terminating his business activity.

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* Memoir prepared by Louis P. Gaston, M. Am. Soc. C. E.

Mr. Safford was notable among engineers of an older generation as one who had special ability as a Locating Engineer at a time when railroad location and construction were at their height in this country. He will also be remembered by his many friends for his genial disposition and for his cheerfulness under a great affliction—deafness.

Mr. Safford was elected a Member of the American Society of Civil Engineers on December 6, 1882.

ARTHUR CHARLES SANDSTROM, M. Am. Soc. C. E.*

Died August 4, 1929.

Arthur Charles Sandstrom was born on April 26, 1888, at Roswell, N. Mex., the son of Joachim Charles Sandstrom and Jacobina Elsa (Funnemark) Sandstrom. Although a fourth generation American, he was of absolutely pure Norwegian strain, his great-grandparents having left Norway to settle in the United States in the early part of the Nineteenth Century.

The story of this remarkable man is one of stern courage and strenuous effort. On the death of his mother, his sole surviving parent, he, at the age of 13, found himself alone face to face with the problems of life. He had to earn his own living, support and educate himself, his three younger brothers, and a sister, as best he could.

In 1906, when 18 years of age, Mr. Sandstrom began his engineering career as Surveyor's Assistant in the employ of the Southern Pacific Railway Company in Central Oregon, which position he retained until the end of 1907. Subsequently, he was engaged as Surveyor and Draftsman on road construction and land surveys in the same State, studying assiduously the while at night and in his leisure through the International Correspondence Schools.

From May, 1908, to September, 1909, he was engaged as Levelman, Transitman, and Chief of Party on land sub-division, extensive topographical surveys, and railway location and construction for the United Railways Company, Portland, Ore.

From September, 1909, until May, 1910, he studied engineering at the Oregon Agricultural College, and, during the Christmas vacation, was Chief of Party on railway location for the United Railways Company. Later, until September, 1910, he worked as Chief of Survey, Draftsman, and Estimator for the same Company.

From October, 1910, until May, 1914, Mr. Sandstrom was both student and Assistant Instructor of Civil Engineering at Leland Stanford Junior University, California. During the summer vacations he was engaged on railway and hydro-electric studies in the Cascade Mountains, in Oregon, in the various capacities of Draftsman, Instrumentman, Chief of Party, Estimator, and Hydrographer. From June, 1914, he worked with the Standard Oil Company on the construction of oil tanks, laying pipe lines, and testing machinery.

He was appointed Resident Engineer in October, 1914, on the location and construction of twenty-five miles of highway which he finished in two years (July, 1916). He also did special work for the Spring Valley Water Company, of San Francisco, Calif., in the calibration of current meters, besides designing and constructing an irrigation system. Subsequently, he held the post of Inspector of Sewerage and Water Supply and other miscellaneous work for the City of Palo Alto, Calif. In this year, 1916, he was graduated as Civil Engineer from Stanford University.

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* Memoir prepared by Robert W. Hornsby, M. Am. Soc. C. E.

South America now offered a wider field for his energies, and in July, 1916, Mr. Sandstrom entered the service of the Braden Copper Company in Chile, as Locating and Preliminary Design Engineer and Estimator for three large hydro-electric projects. He was Engineer-in-Charge over all work at the Pangal Plant (15 000 kw.) for one year, carrying out the construction of roads, housing, excavation of aqueduct, which included fifteen tunnels, and all the accessories of a great establishment. Then, successively, he was for one year, Assistant to the Construction Engineer in special hydraulic studies; in charge of the Materials Department, including requisitioning, tracing shipments, etc., to the extent of $10 000 000 (gold) worth of plant. As is usual in all mining work, his duties were of a most varied nature. He looked after the equipment and supplies; for four months he was Superintendent of Construction (and Manufacture) of 12 km. of 80-in. wood-stave piping (mastic lining), and an 8-mile timber tailings flume; completion of dwellings; operation of the aqueduct for the hydro-electric plant; transport of 15 000 tons of material over difficult mountain roads; and repair of hydraulic works destroyed in storms.

In November, 1919, Mr. Sandstrom transferred his services to the Andes Copper Company at Potrerillos, Chile, as Hydraulic Engineer in direct charge of surveys, designs, and estimates of a 65-km. wood-stave pipe line in the High Andes (La Ola Pipe Line), including the partial excavation of several long hand-driven tunnels. He also made studies of possible hydro-electric plants in the same locality, surveys for a 90-km. railway and aerial tramway at an elevation of 17 000 ft. He was likewise Assistant Superintendent of Construction on preliminary work for large copper reduction plants.

In February, 1920, the Compania Chilena de Electricidad secured his services and Mr. Sandstrom was put in direct charge of the construction of a hydro-electric plant, "Maitenes", and the revision of an already existing plant, "La Florida", totaling 43 375 kw., together with all their accessory works (8-km. aqueduct) which he finished; he also made studies and surveys for future hydro-electric plants on near-by streams. He remained with this Company until June, 1920, having meanwhile solved the difficult problem of eliminating the unusual quantity of sand and gravel carried by the supplying streams.

Mr. Sandstrom's professional services as hydro-electric expert were then requisitioned by the Whitehall Securities Corporation, of London, England, whither he was called to work on designs and estimates for several proposed hydro-electric plants for Chile (22 000 h.p.) from July, 1924, to January, 1925.

Returning to Chile, he assumed the position of Chief Engineer with the Compania Hidro-Electrica del Volcan (a subsidiary company of the Whitehall Securities Corporation), for which he studied several hydro-electrical projects, making final surveys, plans, and estimates for a 40 000 h.p. plant on the Rio Maipo, near Santiago, Chile. He spent five months in England and the United States, completing the plans, ordering construction plant, and making the final arrangements to execute the work (the "Queltehues" Plant).

From April, 1926, until the day of his death Mr. Sandstrom superintended the construction of the Queltehues Plant (Rio Maipo) as Chief Engineer in direct charge of the entire technical construction, and of the commercial phases of the work in Chile. This, his last work, was completed at the time of his death in all but a few minor details. It was particularly difficult, the plant being situated in the upper reaches of the Maipo near its confluence with the Rio Volcan 80 km. southeast of Santiago, and 14 000 ft. above sea level, in the roughest country. The work was extremely heavy, and occupied 2 400 men during approximately 2 years. The aqueduct was 12 km., 3 m., long, 5 166 m. of which was covered, and 2 162 m., open canal. It also included five tunnels with a total length of 3 km., 495 m. The service roadway was 23.5 km. long, with township power houses and proportionate accessories. It is, to date, the largest plant on the Pacific Coast of South America.

On Sunday, August 4, 1929, Mr. Sandstrom was killed by a fall of 65 m., while inspecting a portion of the work. He had been ill in Santiago, but, ever attentive to duty, he rose and accompanied a commission of the management to look over the plant. He was standing on the edge of a chasm leading to the intake, in order to observe better the accumulation of sand, a problem that was always troubling him; the earth gave way, and he was precipitated to the bottom of the chasm and killed instantly.

The news of his death struck Santiago society, both native and foreign, with consternation and grief. His funeral was one of the largest ever seen in the Capital, as if his death had been a National loss (for so, indeed, it was considered), more than 1 000 persons having attended. Such was the end of this great American Norseman.

Mr. Sandstrom was a man of most noble character. There is a trite saying that "there is no greater enemy than one of one's own profession"; but such could never be said of him. He was a born leader (not driver) of men, a just and kindly chief, and a true comrade; hence, he compelled respect and admiration from all with whom he came in contact.

He had a most striking personality, and, as Walt Whitman aptly puts it, "he convinced with his presence." Cast in an heroic mould, of gigantic stature and physique, he worthily represented that mighty race which changed the history of the world, ten centuries ago, and whose descendants to-day form the best elements of English-speaking nations all over the earth.

He was adored by his workmen (for many of whom he had risked his own life on repeated occasions), and by his assistants who looked upon him as upon a father. He handled everything down to the minutest detail in his work, details which others usually leave to their assistants. He was always ready and eager to listen to any suggestion, even from his lowest workman, and to adopt it, did he find it feasible.

In social life Mr. Sandstrom was retiring, and preferred his family and his home which was his Paradise. Although a quiet, proud man, yet he was the essence of kindliness and consideration for others. He took an interest in Freemasonry, and was a member of the American Lodge ("Huelen Lodge") at Santiago.

Mr. Sandstrom possessed all the qualities of a great engineer, and was looked upon as the future organizer of projects of vast proportions embracing the whole of the Andes Range. His death at the early age of 41 years is a serious blow to the Engineering Profession.

In 1920, he was married at São Paulo, Brazil, to Lena Richardson, a native of Manchester, England, who, with four children, survives him.

Mr. Sandstrom was elected an Associate Member of the American Society of Civil Engineers on May 31, 1916, and a Member on July 16, 1928.

LLOYD SCHWARTZ, M. Am. Soc. C. E.*

Died November 7, 1929.

Lloyd Schwartz, the son of John G. and Martha Jane (O'Neil) Schwartz, was born on September 1, 1889, in Golden, Ill. His early education was obtained in the public schools of that town. He was graduated in June, 1911, from the University of Illinois with the degree of Bachelor of Science in Civil Engineering. While at college he was a member of the University baseball team, playing an outfield position regularly. Later, during the second semester of 1913, Mr. Schwartz took post-graduate work in Structural Engineering at the University, and at the same time was an Instructor in Surveying.

Immediately after his graduation in 1911, Mr. Schwartz accepted employment with the Division of Highways of the State of Illinois, and except for the time spent in post-graduate work at the University of Illinois and in the United States Army during the World War, he remained with that organization until his death. His advancement was continuous and in July, 1919, he became District Engineer of the Ninth District, which includes the southernmost fourteen counties of the State with headquarters in Carbondale, Ill. In this position, he directed all State highway activities in the District, including design, construction, and maintenance, and also acted as Consultant in county and township work.

From June, 1918, to June, 1919, Mr. Schwartz was in the service of the United States as a Master Gunner in the 72d Regiment of the Coast Artillery Corps. His overseas service extended from August, 1918, to May, 1919.

He was a member of the Dyhrkopp Engineering Company, a partnership of three, which was formed in October, 1925. He served this Company as Consultant Engineer until its dissolution in 1927. A few months after the Company was formed, he applied for, and after an examination, received his license as a Structural Engineer.

Mr. Schwartz was active in many civic, fraternal, and social enterprises. He held membership in the Carbondale Rotary Club, the American Legion, the Benevolent and Protective Order of Elks, the Masonic Blue Lodge, the Chapter, the Commandery, and Shrine, the Jackson Country Club, the Midland Hills Country Club, and the Dog Tooth Bend Hunting Club.

Lloyd Schwartz was a clean and ardent sportsman with marked ability in many forms of athletics. However, his recreational activities indicated a preference for golf in the summer and hunting in the fall and winter; and it was while hunting ducks at the Dog Tooth Bend Hunting Club on the Mississippi River that he met his death. Late in the afternoon of November 7, 1929, he had shot a duck which fell into the river a short distance from the blind. In attempting to retrieve it, he waded off a shelf in the river bed into deep water, and the weight of his hunting togs gave him no chance for his life.

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* Memoir prepared by E. R. Knight, Assoc. M. Am. Soc. C. E.

Funeral services were held at his home in Carbondale, Ill., under the direction of the Knights Templars of Beauseant Commandery, after which interment ceremonies were completed in Golden, with the Masonic Blue Lodge of that town in charge.

He was married on February 14, 1920, to Lucinda A. Wesson, at Aurora, Ill., who survives him, together with three sisters, Mrs. Glen Cutforth, of Eldorado, Kans., Mrs. Laura E. Duff, of Cedarville, Wis., and Mrs. May Reitman, of Chicago, Ill., and two brothers, Earl Schwartz, of San Francisco, Calif., and Clyde Schwartz, of Quincy, Ill.

Mr. Schwartz was elected an Associate Member of the American Society of Civil Engineers on March 12, 1918, and a Member on March 15, 1926.

CHARLES ALFRED SHEPPARD, M. Am. Soc. C. E.*

Died October 18, 1928.

Charles Alfred Sheppard was born at Southampton, England, on February 12, 1866, the son of George and Ellen (Major) Sheppard. In 1870, the family came to the United States and settled at Beatrice, Nebr.

Mr. Sheppard began his engineering career by employment in minor capacities, on railroad location surveys and construction work during the great development of the West in the Eighties. At that time he was engaged in work for various railroads which have since become part of the Chicago, Burlington and Quincy Railroad System. In 1888, he devoted a year to special study at Fairfield College, Fairfield, Nebr., and, in 1889, became Assistant Engineer in charge of second-track construction for the Louisville and Nashville Railroad Company.

An extensive industrial development was started at Granite City, Ill., in 1892, and Mr. Sheppard was placed in charge of the field work, supervising the construction of the sewer system, water-works, streets, paving, etc. To a large extent, this undertaking proved to be the turning point in his career, as from that time on practically all his work was in connection with the growth of the St. Louis Industrial District on the east side of the Mississippi River in the construction of railroads, levees, sewers, drainage systems, and highways. This work grew to such an extent that, in 1900, he formed a partnership with Mr. W. H. Morgan, with offices at Edwardsville, Ill. This partnership continued until his death.

When the construction of the Chain of Rocks-Kingshighway Bridge across the Mississippi River was begun in the fall of 1927, Messrs. Sheppard and Morgan were retained to supervise the work on the east side of the river, and Mr. Sheppard devoted most of his time to this work until October, 1928, when he took a short vacation to visit a sister in Omaha, Nebr., who was ill. As he was preparing to return from this trip on October 18, his automobile was struck by a heavy truck and he was instantly killed.

Like many other engineers, Mr. Sheppard was very modest as to his ability. He was held in high esteem by all who knew him. One of his principal pleasures was derived from his membership in the choir of St. John's Methodist Episcopal Church of Edwardsville. His true character as a man cannot be better expressed than in quoting from a resolution prepared by the Official Board of that Church:

"He had profound and clear Christian convictions, and translated them into the practices of his daily life. All his conduct was a practical application of his religious principles. Recognizing that beliefs are individual, he did not attempt to impose his beliefs or rules of conduct on others, and could hold and express differences of opinion without giving or taking offense. Capacity for integrity, sound judgment, and industry, a talent for friendship, never failing good humor, are qualities he shared with many; few are as faithful in daily life to lofty spiritual ideals.

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* Memoir prepared by Baxter L. Brown, M. Am. Soc. C. E., and W. H. Morgan, Esq., Edwardsville, Ill.

"The close of his life, like the setting sun, leaves to the world the glow of perfect day.

"Walk with him and keep unbroken
The bond which Nature gives,
Thinking that our remembrance, though unspoken,
May reach him where he lives."

In 1895, Mr. Sheppard was married to Olive Morgan, of Bostwick, Nebr., who died on June 25, 1927. Three children survive them: Leila, Robert, and Charles H. The latter succeeded his father in the re-organized firm of Sheppard, Morgan, and Schwab, with offices at Edwardsville and Alton, Ill.

Mr. Sheppard was elected a Member of the American Society of Civil Engineers on November 30, 1909.

MERRITT HAVILAND SMITH, M. Am. Soc. C. E.*

DIED DECEMBER 9, 1926.

Merritt Haviland Smith, the son of Merritt Haviland and Mary Adelia (Howard) Smith, was born in New York, N. Y., on May 21, 1862. During his attendance at the Pennsylvania Military College, from which he was graduated in 1880 with the degree of Civil Engineer, he received both engineering and military instruction.

After his graduation, Mr. Smith studied law for a year, and then entered the Engineering Corps of the Pittsburgh and Western Railroad Company, with which he remained until 1884. On leaving the railroad service he was employed by the City of New York in the Department of Public Works on the Croton Aqueduct until 1890. For approximately two years thereafter, he was engaged on general engineering and land development work in Florida.

In 1892, Mr. Smith re-entered the service of the City of New York in the Department of Finance, continuing in that Department until 1906. He was then appointed Department Engineer in the Board of Water Supply in charge of the Southern Aqueduct Department, and, in 1910, promoted to the position of Deputy Chief Engineer. On August 1, 1914, he was appointed Chief Engineer of Water Supply in the Department of Water Supply, Gas, and Electricity, and continued in this position until he was retired on December 3, 1926.

In Mr. Smith's engineering association with the City of New York, which covered a period of about forty years, the major portion of his service was in connection with its water supply system; but while he was in the Finance Department, he also was called upon to do important work in connection with virtually all the engineering activities of the City. As Department Engineer in charge of the Southern Aqueduct Department, he was responsible for the construction of the Catskill Water Supply System from the Hunter's Brook Siphon to the city line, including the Kensico and Hillview Reservoirs. From 1910 to 1914, his work on the Catskill System was mainly of an administrative character. As Chief Engineer of Water Supply he had the responsibility of maintaining and operating the entire water supply system of the City of New York, including extensive construction work connected therewith.

In addition to his engineering activities, the military career of Mr. Smith stands out for the extended service which he rendered to his State and Country. This military service started when he joined Company F of the 7th Infantry, New York National Guard, on October 19, 1880; he was appointed Corporal on March 2, 1886; and was honorably discharged on September 19, 1889. He again enlisted as a private in Troop 1, Squadron A, N.Y.N.G., on January 2, 1896; was promoted to Corporal, April 18, 1896; Sergeant, April 27, 1899; and honorably discharged on January 1, 1902. On January

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* Memoir prepared by William W. Brush and Jules Breuchaud, Members, Am. Soc. C. E., and Robert Ridgway, Past-President, Am. Soc. C. E.

6, 1902, he was commissioned a Lieutenant of Squadron A; promoted to the rank of Captain on February 3, 1906; and resigned on April 24, 1907. He was commissioned as Captain of the First Cavalry, N.Y.N.G., on February 7, 1912; promoted to the rank of Lieutenant-Colonel on May 23, 1912; appointed Lieutenant-Colonel of the First Field Artillery, N.Y.N.G., October 19, 1914; advanced to the rank of Colonel on March 23, 1917; and was honorably discharged from the military service of the State, while in the United States Army, in 1919.

Colonel Smith's service with the United States Army shows that he was commissioned by the President on June 7, 1898, as Captain, for service in the Spanish-American War, and was assigned to Company D, First United States Volunteer Engineers. His regiment was part of the force sent to Porto Rico, and he was commissioned Major of the regiment on January 24, 1899, and honorably discharged, January 25, 1899, on the mustering out of the regiment on that date. He was commissioned a Lieutenant-Colonel, Field and Staff, First Field Artillery, N.Y.N.G., October 20, 1914; and was called into the Federal Service on June 28, 1916, when his regiment formed part of the force sent to the Mexican Border that year. He was mustered out of the Federal Service on November 15, 1916, and again mustered into the Federal Service on June 22, 1917, for the World War. He left the United States for overseas' service, June 30, 1918, and was actively engaged in the field with his regiment, the 104th Field Artillery, returning to the United States on January 1, 1919. He was honorably discharged on February 12, 1919.

The character of Colonel Smith's military work in the World War is shown by the citation that he received in Special Order No. 86, 27th Division, 1919, for exceptional devotion to duty during the operations of the 52d Field Artillery Brigade in the vicinity of La Claire Farm, and Bois de Forges, France, on September 9 to October 18, 1918, before and during the Meuse-Argonne Offensive. The War Department record further states: "This officer rendered conspicuous services, and although seriously ill, continued in the performance of his duties until completely prostrated."

Although Colonel Smith was physically unable to continue actively in the military service after his return from France, he maintained a live interest in its work to the time of his death.

In addition to his engineering and military service, Colonel Smith took part in the official work of the City of Yonkers, N. Y., of which he was a resident, acting as Park Commissioner from 1900 to 1902, and as Police Commissioner during 1905 and 1906.

No description of Colonel Smith's life would be complete without special reference to his exceptional ability as a leader in any gathering. In height he was several inches above his fellows, with a fine military bearing. His voice in song or story would hold the attention of all, and added greatly to the enjoyment of those present at the many social and other functions that he attended. As a result of his varied activities, his friends were numbered literally in the thousands.

On December 30, 1890, he was married to Maude M. Underhill, by whom he is survived. His two children were Merritt H., Jr., born in 1891, who died during the World War while at the Plattsburg, N. Y., Training Camp, and Roderick Ray, who died in childhood.

He was a member of the American Water Works Association, the Municipal Engineers of the City of New York, the American Society of Mechanical Engineers, and the Chamber of Commerce of the State of New York.

Colonel Smith was elected a Junior of the American Society of Civil Engineers on June 5, 1889, and a Member on April 3, 1907.

GEORGE FETTER STICKNEY, M. Am. Soc. C. E.*

Died January 27, 1929.

George Fetter Stickney was born on January 24, 1869, in St. Paul, Minn., the son of Amos Stickney and Virginia (Fetter) Stickney. His boyhood days were spent in various cities where his father, who was a Major and later a Brigadier General in the Corps of Engineers, United States Army, was stationed in charge of river and harbor improvements. He received the degree of Bachelor of Philosophy from the Sheffield Scientific School of Yale University in 1891, where he was a member of Chi Phi Fraternity.

Mr. Stickney's first employment was with the Mexican International Railroad Company. From 1891 to 1893, he served as Rodman, Leveler, and Resident Engineer retracing location surveys on the Durango Extension, supervising the construction of 30 km. of line, including yards and freight and passenger depots in the City of Durango, Durango, Mexico, making reconnaissance and preliminary surveys to Rio Tunal and supervising the grading of 25 km. of the Sierra Mojada Extension.

He entered the Government Service in 1893 as United States Assistant Engineer in charge of repairs on locks and dams on the Kentucky River in Kentucky, and made surveys for the location of Lock No. 7 at High Bridge, Ky., building the lock-gates and designing boats, barges, dwellings, etc. From 1895 to 1898, he was in local charge of the operation and care of five locks and dams, a dredge and a snag-boat on Green and Barren Rivers and a lock on Rough River, Kentucky. In addition, he rebuilt part of Dam No. 2 and finished the construction of Lock No. 2 at Rumsey, Ky.

Mr. Stickney served for nearly two years in the United States and Cuba during the Spanish-American War as Captain in command of Company C, Third Regiment, United States Volunteer Engineers. In 1899 and 1900 he made a survey for the location of a terminal railroad at Sabine Pass, Tex.; graded streets on the outskirts of Paducah, Ky.; and made plans for the improvement of Jefferson Barracks, Missouri, with layouts for sewers, retaining walls, roads, and the grading of the parade grounds.

In 1901 he made the surveys and designed the masonry for a bridge for the Mississippi River and Bonne Terre Railroad Company in Southeastern Missouri. In 1901 and 1902 he had charge as Assistant Engineer, for the Lake Superior Power Company, of constructing the "compensating works", a movable dam of five large Stoney gates, at the head of the rapids in St. Mary's River, at Sault Ste. Marie, Ont., Canada. He also superintended the construction of piers and abutments of the Louisville and Nashville Railroad Bridge across the Alabama River at Selma, Ala., for Paterson, Collier and Company, Contractors.

In 1903, Mr. Stickney superintended the construction for the J. S. Paterson Construction Company of a concrete arch viaduct forming the east approach to the Southern Illinois and Missouri Bridge, at Thebes, Ill. In 1904 and 1905, as U. S. Assistant Engineer, he was employed on harbor work in

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* Memoir prepared by Joseph Ripley, M. Am. Soc. C. E.

Northern New Jersey and along Long Island Sound. For nine years, from 1905 to 1914, Mr. Stickney was employed in the Department of the State Engineer and Surveyor of New York, as Supervising Engineer, in connection with the Barge Canal; and in addition he had general charge of the design of large sections of the New York State canals, namely, the Erie, Champlain, Oswego, and the Cayuga-Seneca Canals, together with the supervision of construction on more than 250 miles of waterway, including approximately one-third of all the canal structures, the construction cost of which exceeded $35 000 000.

Another important engagement was that of Consulting Engineer to the Lake Erie and Ohio River Canal Board of Pennsylvania from 1915 to 1917. As such Mr. Stickney had charge of preliminary investigations, surveys, preparation of plans, estimates, etc., for the proposed waterway which was estimated to cost $100 000 000. This latter work included the canalization of the Beaver and Mahoning Rivers, in Pennsylvania and Ohio, and the construction of a canal from Niles, Ohio, to a point on Lake Erie just west of Ashtabula, Ohio, with locks, dams, and reservoirs, as well as port developments at Lake Erie and at various cities along the route of the canal.

As a Consulting Engineer Mr. Stickney maintained an office in Albany, N. Y., from 1917 to 1928, and made many hydraulic investigations for water-power developments and river improvements. He also was an expert witness in several claim cases involving water rights. He was the inventor of the siphon spillway for discharging water through dams, and of an automatic movable crest for dams. He contributed a paper entitled "Siphon Spillways"* to the *Transactions* of the Society, as well as articles on engineering subjects to various technical publications.

Mr. Stickney was a man of keen intellect and had the faculty of quickly sifting out the essential elements of a problem and the ability to work out in detail an accurate solution. His skill in sketching rapidly and accurately with a few lines materially aided in making plain his conception of the subject or of an object. He was very methodical and kept his records in precise and complete form. He was self-reliant and on occasion was tenacious in holding to his views. Always he was a thorough gentleman, with an ever-present quiet dignity. He had been in declining health for three or four years before his death, the cause of which he had attributed to heart trouble, not knowing he had cancer of the stomach. He was buried at West Point, N. Y.

In 1894 he was married to Katherine Halloran, of Frankfort, Ky., and is survived by his widow, two children, Josephine Stickney and George Fetter Stickney, Jr., and by his mother and two sisters, all residing in New York, N. Y.

Mr. Stickney was elected a Junior of the American Society of Civil Engineers on May 1, 1894; an Associate Member on April 5, 1899; and a Member on September 4, 1906.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 1098.

EDWARD AHLERT STUHRMAN, M. Am. Soc. C. E.*

Died May 31, 1929.

Edward Ahlert Stuhrman was born in New York, N. Y., on December 18, 1884, where his early life was spent and where he prepared himself for college.

He was graduated from Columbia University in the Class of 1907, with the degree of Bachelor of Science in Civil Engineering. Later, he was granted a license by the State of Illinois to engage in the practice of Architecture and was a Registered Engineer in the State of Florida.

After his graduation, Mr. Stuhrman was employed for a short time as Timekeeper and Foreman, by the Carlin Construction Company, of Brooklyn, N. Y. He then entered the service of the Turner Construction Company of New York, as Draftsman and Assistant Engineer, resigning to become connected with the Industrial Engineering Company of New York.

Upon leaving New York, he went to Chicago, Ill., where he was employed by the Chicago, Milwaukee and St. Paul Railway Company, serving with that Company for several years. From Chicago, Mr. Stuhrman went to Baltimore, Md., with the Arthur Tufts Company, and was in charge of the design and construction of warehouses and concrete water tanks for the Coco Cola Company at Baltimore, Los Angeles, Calif., New Orleans, La., Kansas City, Mo., Winnipeg, Man., Canada, and Atlanta, Ga. While in Atlanta, he was engaged as Engineer on the Candler Warehouse, Emory University Buildings, and Wesley Memorial Hospital.

Mr. Stuhrman moved to Miami, Fla., in 1924 and during the few years of his residence there, he was connected with some of the largest construction projects in Southern Florida. Some of the more important buildings on which he served as Engineer are the Congress Building, Seybold Building, First Trust and Savings Bank Building, City National Bank Building, Miami Senior High School, Gulf Stream Apartments, at Miami Beach, Whitehall Hotel, Palm Beach, Rolyat Hotel, at St. Petersburg, and the passenger station and hangars for the Pan-American Railways at Miami.

Mr. Stuhrman was a member of St. George's Protestant Episcopal Church, of New York. He was a Thirty-Second Degree Mason and a member of Yaraab Temple, A. A. O. N. M. S., of Atlanta, Ga. At the time of his death, he was President of the Miami Engineers' Club. He was also a member of the American Concrete Institute, and the Kiwanis Club.

In Mr. Stuhrman's untimely death, Florida has lost one of her most brilliant adopted sons.

He was married in Brooklyn, N. Y., on August 26, 1908, and is survived by his widow, Violet L. Stuhrman, and two sons, Everard and Ahlert.

Mr. Stuhrman was elected a Member of the American Society of Civil Engineers on September 10, 1918.

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* Memoir prepared by Meldrim Thomson, Esq., Miami, Fla.

CHARLES FREDERICK TAYLOR, M. Am. Soc. C. E.*

DIED APRIL 14, 1927.

Charles Frederick Taylor was born on March 27, 1860, at Lewiston, Ill. He was but a little lad when his family moved to Mooers, N. Y., a small town in the northern part of the State near the Canadian Border. Here he received his primary education and later, was prepared to enter Syracuse University, at Syracuse, N. Y.

Many amusing incidents are told of Mr. Taylor's early boyhood in the society of his several brothers and sisters. At the age of four, he had occasion to set forth on his first journey by train, going by way of the "Grand Trunk." He presented a very sad figure when through disobedience he lost his hat out of the car window and for penance was made to wear his sister's bonnet "home to grandpa's".

In 1880, Mr. Taylor and his brother, Henry L. Taylor, entered Syracuse University, where Mr. Taylor was enrolled for three and one-half years as a student in engineering. He was graduated in the Class of 1884 with the degree of Bachelor of Civil Engineering, and also with some months of special work at Rensselaer Polytechnic Institute, Troy, N. Y., to his credit. He received an honorary Civil Engineering Degree from his Alma Mater in 1905. He was affiliated with both the Phi Delta Theta and Sigma Psi Fraternities.

Mr. Taylor's active professional career included a wide variety of positions and great expanse of territory. The majority of these engagements are given in the following brief survey: From July to November, 1883, as Chainman and, afterward, Rodman and Leveler, on the construction of an extension of the Portland and Ogdensburg Railroad; from July to November, 1884, as Transitman on the construction of the Canada Atlantic Railroad; from May, 1885, to May, 1889, with Moffett, Hodgkins, and Clarke, on work including preliminary surveys of Grenell Island Park near the Thousand Islands and surveys at Oneida Lake for the Central City Water-Works Company of Syracuse; from September, 1885, to February, 1886, as Assistant Superintendent of Construction for the Newark, Ohio, Water-Works; from May, 1886, to January, 1887, as Construction Engineer on the Kankakee, Ill., Water-Works; from January to December, 1887, as Inspector of pipe laying and similar detail at the Canada Pipe Foundry for the Belleville, Ont., Canada, Water-Works; from April to December, 1888, as Construction Engineer for the Berlin, Ont., Water-Works and, later, with the Coburg, Ont., Water-Works. From May, 1889, to January, 1890, he served as Division Engineer on the construction of the Kinderhook and Hudson Railway from Niverville to Stuyvesant Falls, N. Y.; and after January, 1890, as Construction Engineer for the entire line as far as Hudson, N. Y.

In October, 1890, Mr. Taylor entered a field of service which gave him, personally, wide and rich experience in his chosen profession. He was engaged for six years with the Syracuse Water Board in charge of surveys at Skaneatles Lake and along the Skaneatles Outlet, and also on surveys connected with the condemnation proceedings to acquire the plant of the Water Company, water-power rights, etc. Early in his association with the Board he became Engineer in Charge of City Distribution of the Water-Works of the City of Syracuse. His faithful service in this employ put him in touch with financial backing that was in itself rare experience. During the last year of his service (1896), he personally supervised the laying of 80 miles of water mains.

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* Memoir prepared from information supplied by Dr. Henry L. Taylor, University of the State of New York, Albany, N. Y., and Paul P. Wiant, M. Am. Soc. C. E.

From 1897 to 1903, Mr. Taylor acted in the capacity of Contractor for New York State on canal construction. During the latter part of this period, he also served as Treasurer of the Boston Tunnel Construction Company and probably the most noteworthy of his engineering feats was Section B on the East Boston Tunnel—more than a mile in length under the harbor. The tunnel was constructed "101 ft. below mean-tide level, the side walls and arch of concrete in the largest span ever constructed at that date in soft materials." It is remarkable that there occurred "the loss of but two human beings at a time when work was abandoned on the New York North River and the dead had not been removed from the flooded works." Also, at this time, the anthracite coal strike threatened alarming and continued financial disaster.

His Boston subway experience so fitted Mr. Taylor for under-water work, that he became Engineer or Treasurer of a dredging company operating in the Hudson Valley. The assembled plant made possible the successful bids for about one-third of the work of enlarging the New York State Canals under an appropriation of $9 000 000. In 1905, Mr. Taylor served as Treasurer and Chief Engineer of the Seely-Taylor Company, Engineers and Contractors, on a contract for the 30-in. submerged, cast-iron pipe sewer at New Rochelle, N. Y. At the conclusion of this work, he became President of the Maritime Dredging Company of New York, and was engaged on marine dredging work in New York Harbor until the close of the World War. His headquarters in New York demanded the presence of his family in the vicinity, and, after a few years in the city, he moved to New Rochelle where he resided until the death of his wife in the winter of 1918.

After a brief rest at Clifton Springs, N. Y., Mr. Taylor again took up his work, specializing in coal investigations, and, in this connection, he made extensive trips through the mining centers of the United States. In February, 1919, he began the settlement of his affairs and at a time of life when most men do not plan to undertake new and untried tasks, he offered himself to the Board of Foreign Missions of the Methodist Episcopal Church for service in any foreign field where he might be needed for building or other technical work. He was accepted early in 1922 for a short term of service in Foochow, China. The primary purpose of this engagement was to cover furloughs of the then existing staff of the Fukien Construction Bureau, an organization for doing professional work in the Fukien Province of China.

Within a month after his arrival in Foochow, he was given full charge of an extensive building program, which he was able to manage very successfully, due to his extremely broad experience in the handling of materials and men.

He constructed several notable buildings in this far distant country, which will stand for generations as a monument to his energy.

Mr. Taylor's term was to have been for three years, but this was extended to five, and would have been longer had it not been for local conditions which make all forms of building work dwindle to insignificant proportions during various intervals. He spent his month's vacation annually in travel. The first journey in 1923 in Northern China covered 3 000 miles; the second in 1924 through China, Korea, and Japan, 6 000 miles; the third in 1925 to Hongkong and The Philippines, 2 500 miles; and the fourth in 1926 to Chung King, West China, 4 000 miles.

When his work was completed in Foochow in December, 1926, he set out on his fifth and last journey, which was to be a leisurely trip around the world, with the intention of stopping at whatever place he might find something useful and worth while to do. He had traveled through the Straits Settlements, India, many of the East India Islands, Australia, and parts of South and Central Africa, when he was overtaken by illness at Old Umtali, in South Rhodesia. The end came very suddenly and peacefully, on August 14, 1927, while he was sitting in a chair reading a paper, apparently without the least premonition that death was near. His illness, which he thought to be only slight, proved to be myocarditis, and was fatal. A few days before he died, he had received a cable asking him to proceed to Monrovia, Liberia, to construct a school building for the Methodist Mission Board, which he was preparing to accept.

Mr. Taylor was a good business man and very honorable and upright in all his dealings. He was most painstaking and thorough, and never considered his work finished until it was done right. He had many of those qualities of geniality and brotherliness which made his friends think highly of him. By his outstanding character he made a very strong impression on the Chinese community in which he had lived for five years.

A letter from Mr. R. C. Gates, of Old Umtali, to Dr. Taylor, of Durban, South Africa, his nephew, bears the following message which tells of the affectionate memory in which Mr. Taylor is held:

> "Our hearts were very heavy over the sudden death of our brother missionary who had so recently come among us, and who had so won our affection and esteem. Among other traits, I especially admired his high code of honor. But we all felt glad that since he died on his journeys that he at least could die among friends and be buried on mission land where his grave can be looked after and his name kept in memory."

Mr. Taylor was married on March 27, 1888, to Emma R. Gibbs, of Hamilton, Ont., Canada, whose death preceded that of her husband. He is survived by two brothers, the Rev. B. S. Taylor, of Los Angeles, Calif., and Dr. Henry L. Taylor, of Albany, N. Y., and two sisters.

Mr. Taylor was elected an Associate Member of the American Society of Civil Engineers on September 4, 1895, and a Member on April 4, 1905.

ALEXANDER MILLER TODD, M. Am. Soc. C. E.*

Died September 11, 1929.

Alexander Miller Todd, the son of George T. and Marion (Miller) Todd, was born September 28, 1874, in Rankin County, Mississippi. His early education was obtained in the public schools of Rankin County. He matriculated at Texas Agricultural and Mechanical College in September, 1890, and was graduated in June, 1894, with the degree of Bachelor of Civil Engineering.

On the completion of his college course, Mr. Todd entered the service of the Mississippi River Commission, Third District. He started as a Rodman at the bottom of the Engineering Department, and by hard work advanced rapidly through the grades of Levelman, Transitman, and Inspector, to become a Superintendent of Construction in 1900.

His early survey work brought him into intimate field contact with the Mississippi River and gave him a practical background that proved invaluable later when he reached a position to study the problems of river control. From the first, he manifested a lively interest in the behavior of the Mississippi, and his keenly analytical mind early began to consider the difficult problems of controlling the mighty "Father of Waters".

Mr. Todd devoted his whole professional life to the Federal Service. From a Superintendent of Construction in 1900, he rose steadily through the various grades in the Engineer Department, until at the time of his death he held the position of Senior Engineer. His entire service was in what is now called the Vicksburg Engineer District.

During the war-time emergency, from 1917 to 1919, he served as District Engineer of the Third Mississippi River Commission District and directed, during this trying time, the prosecution of many important works. His services were so highly regarded by the Federal Government that he was made a Special Disbursing Agent, United States Engineer Department, U. S. Army.

Mr. Todd was a practical student of many of the important flood-control problems on the Mississippi River. His studies of gauge relations were most exhaustive and were used extensively in determining levee grade lines and in predicting flood crests. He gave much time to the study of bank protection works, and his influence was felt, not only in the theoretical analysis of this problem, but in the practical design of machinery for placing revetment. In levee building, also, he became an authority. In the District Office his opinion, as to the relative merits of revetment or levee construction in the solution of a particular local problem, was the deciding factor in making the recommendations for construction.

In 1913, when it became apparent that a much extended program for Mississippi flood control would be necessary, Mr. Todd foresaw that the available supply of material for the older types of bank protection works or revetment would be insufficient to meet the demands. The use of concrete was coming into prominence, and it was soon realized that this offered a possible solution of the problem. When the articulated concrete mat was developed, Mr. Todd designed an all-steel floating plant for the manufacture and placing of this mat, which proved most successful. This plant was unique, in that it was equipped with the first known concrete mixer utilizing a horizontal distributing boom.

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* Memoir prepared by Elliott J. Tucker and George R. Clemens, Associate Members, Am. Soc. C. E.

The demands of the 1914 Mississippi River Commission program for increased height and section of levees were such that it was apparent to the levee student that improved methods of construction would be necessary. Accordingly, Mr. Todd devoted much of his time and energy to the study of more economical methods for placing earthwork. Early experiments, with a taut-line cableway machine, were not as satisfactory as desired. Further experiments with a slack-line drag-bucket cableway were more successful and, in 1916, in collaboration with engineers of the Bucyrus Company, he designed the tower levee machine embodying this principle. This machine proved very successful and has been used with but very little change in the original design to the present time (1929). The capacity of these machines is approximately 150 000 cu. yd. of earthwork per month—something impossible with the older type of equipment.

In addition to his work on the main Mississippi River, Mr. Todd directed certain surveys on the tributaries. In 1910, he made the field survey and report for the power development on the Upper Ouachita River, near Hot Springs, Ark., out of which has developed the project (Federal Power Commission No. 271) for three power dams—Remmel (built), Carpenter (under construction), and Blakely Mountain (proposed).

Personally, Mr. Todd was held in the highest esteem. At more than one heated conference his was the quiet head that calmed the discussion and brought order and progress out of argument. His ability to serve in difficult capacities was attested by the fact that he was appointed in May, 1925, by the President of the Mississippi River Commission to serve as a member of the Investigating Board of the *Norman* steamship disaster at Memphis, Tenn.

He was keenly conscious of his civic duties and served in many capacities in a number of organizations. He was a leader in community betterment and devoted much of his time to these interests.

Death came to Mr. Todd very suddenly, while on an inspection of the Carpenter Dam construction near Hot Springs, Ark. His life was cut short by apoplexy at the crest of his active career. Funeral services were held at the First Baptist Church, Vicksburg, with interment at the City Cemetery. He is survived by his widow, Sue C. (Carleton) Todd, to whom he was married May 17, 1898, in Chicot County, Arkansas, and by three children—Carleton R. (Lieutenant, U. S. Navy), Marion E. (Todd) Rogers, and Elizabeth A. (Todd) Burnett.

Fitting testimonial to Mr. Todd's many sterling qualities was given in the following public announcement of his death, made by the District Engineer, Major John C. H. Lee, Corps of Engineers, U. S. A., M. Am. Soc. C. E.:

"With deepest regret the District Engineer announces the death of Sr. Engineer Alexander Miller Todd, resulting from a stroke of apoplexy while inspecting the construction work at Carpenter Dam, Ouachita River, Arkansas, about 11 A. M., September 11, 1929.

"Mr. Todd died on duty, ending a career of outstanding merit and remarkable service to the United States as well as to the people of the valley. In this career of devoted and unsurpassed service, Mr. Todd identified himself with all phases of Mississippi flood control. He has not only been an invaluable student of great flood control problems, but he has exemplified a practical ability to execute work and hold the devoted loyalty of his fellow employees.

"Mr. Todd was one of the senior members of the American Society of Civil Engineers in Mississippi; an active member of the Vicksburg Chamber of Commerce and the Vicksburg Rotary Club. He was a Deacon of the local Baptist Church, a Director of the Y. M. C. A., and has held the honored respect of the entire community. To all who have known him, Sr. Engineer Todd's life has been an inspiration. Revered, admired, beloved by his fellow men, this able public servant and loyal citizen of a bereaved community will live always in the memory of his friends."

Mr. Todd was elected a Junior of the American Society of Civil Engineers on October 1, 1895; an Associate Member on October 2, 1901; and a Member on June 6, 1905.

JOB TUTHILL, M. Am. Soc. C. E.*

Died December 28, 1928.

Job Tuthill was born, of Puritan stock, in Blooming Grove, N. Y., on October 26, 1855, the son of Benjamin S. and Mary Elizabeth (Cooley) Tuthill.

He was graduated from the University of Michigan, at Ann Arbor, Mich., in 1883, with the degree of Bachelor of Science in Civil Engineering. He at once entered railway service as a Civil Engineer and continued actively in this work for the remainder of his life. His ability as a designer and builder of railway structures came to be widely recognized, chiefly through his work on the Pere Marquette Railway. He served this Company and its predecessor lines as Assistant Engineer, Bridge Engineer, Assistant Chief Engineer, and Chief Engineer, during a total of thirty-five years.

Mr. Tuthill was a constant and thorough student of engineering subjects. He took an active interest in the work of the various engineering societies of which he was a member, and gave liberally of his time to the work of their committees. He possessed, in full measure, the engineer's habit of mind, viewing his work as the fulfillment of a responsibility to his employers and his countrymen, to build safely and well, and to make proper use of, and conserve, Nature's materials. His conscience is reflected in the hundreds of well-designed and carefully executed structures standing as monuments to his faithful labor.

His distinguishing quality was his fine character. He will be remembered for his gentlemanly, courteous manner toward his associates, his loyalty to his profession and his friends, his modesty, reticence, quiet speech, unbounded sympathy, and his charitable attitude toward human weakness or misfortune. He was ever ready to aid those in need of moral support or material assistance.

During his professional life many young men passed under his tutelage. These were uniformly influenced to their lasting benefit—in their attitude toward their work, by Mr. Tuthill's careful instruction, and, in their characters, by his fine example. In 1914, his College conferred upon him the Honorary Degree of Master of Engineering. Dr. Hutchins, in bestowing the degree, eulogized Mr. Tuthill as follows:

> "A graduate of the University of Michigan, Department of Engineering, in the Class of 1883. Prominent as a builder of railway bridges and terminals, he has won praise for his work and respect for his modest manhood."

In 1884, Mr. Tuthill was married to Florence B. Craig, of Kalamazoo, Mich., by whom, with a daughter, Louise Kingsbury, he is survived. A son, Benjamin Sayre, died in 1910.

Mr. Tuthill was elected a Junior of the American Society of Civil Engineers on June 6, 1888, and a Member on October 4, 1893.

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* Memoir prepared by the following Committee of the Detroit Section: Charles S. Sheldon, *Chairman*, J. F. Deimling, H. E. Riggs, Members, Am. Soc. C. E., Paul Chipman, Assoc. M. Am. Soc. C. E., and A. L. Grandy, Esq., Detroit, Mich.

CHARLES CLIFTON UPHAM, M. Am. Soc. C. E.*

DIED JANUARY 7, 1928.

Charles Clifton Upham was born on May 5, 1852, of good old New England stock. He was of the seventh generation, in direct line of descent, from John Upham who came from England in 1635 with a colony of twenty-one families under the leadership of one, Joseph Hull. They settled in what was then called Wessaguscus, afterward named Weymouth, in Massachusetts. John Upham was evidently a man of consequence in his community having been a Delegate to the General Court as well as Selectman of the Town. He was one of six who treated with the Indians for the lands of Weymouth, the settlers previously having occupied this site as squatters. Later, in Malden, Mass., to which he soon moved, John Upham served as Selectman as well as Moderator of the Town Meeting. His son, Lieut. Phineas Upham, who gained his title in the Colonial Wars, was also a man of prominence. It was said of him that "Worcester [Massachusetts] owes its foundation in no small degree, as it appears, to his activity and energy."

Other Uphams in the direct line of descent appear to have been men of prominence and enterprise. Several of Mr. Upham's ancestors, not of that name, however, came to Massachusetts in the early half of the Seventeenth Century. His father's mother was a *Mayflower* descendant. His father, born in Castine, Me., had moved to Boston, Mass., and was one of a party of ten to sail from there for California, around Cape Horn, in 1849. After eighteen months, he returned and lived in Woburn, Mass., where Charles Clifton Upham was born. In 1853, the family moved to Dixon, Ill., where young Upham was brought up. The father was recognized in Dixon as a man of education and of excellent literary taste. Charles Clifton Upham had a fine family heritage.

Beginning at eighteen years of age, from 1870 to 1872, Mr. Upham was Assistant Engineer on the Illinois River Improvement, Slackwater Navigation. The next three years found him gaining his first experience in railroading as Assistant Engineer of the Grand Junction Railroad and the Belleville and North Hastings Railroad, in Ontario, Canada. Then, for three years he was in Del Norte, Colo. During at least part of this time, he was expecting to install lixiviation works for reducing ores, but this project never materialized.

Meanwhile, he qualified as a United States Deputy Mineral Surveyor. He enjoyed relating that, while he was in Del Norte awaiting developments, he and a friend, who later became Cashier of the local bank, worked with pick and shovel on highway construction, and that he loaned the contractor money when he needed it to complete his contract.

For several years Mr. Upham was with the Atchison, Topeka, and Santa Fé Railroad Company on location and construction, having served at one time as Transitman on part of the difficult location of the line through the Grand Canyon of the Arkansas. When the Mexican Central Railroad was started from Paso del Norte (now Juarez) south toward Chihuahua, he became

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* Memoir prepared by C. Frank Allen, M. Am. Soc. C. E.

Locating Engineer, there being then no Chief Engineer. After several hundred miles had been located, he was offered the berth of Chief Engineer at the same salary, $250 per month. This he refused as he deemed the salary inadequate. He was much needed on the work and did not hesitate to remain as Locating Engineer, the salary being then, as he thought, adequate for that position. This was an example of the refusal of desirable advancement because of an honorable respect for the dignity of his profession—and this on the part of a man not above working with pick and shovel, at a time when he had ample funds in his pocket. Mr. Upham needed no formal code of ethics; instinctively, he steered straight.

By that time he had acquired a comfortably ready use of the Spanish language and was very popular with the various Mexican officials at Paso del Norte. He was conspicuously an asset to the railroad. It was at this time that his locating party was wiped out by a band of Yaqui Indians on the war-path, and it was first thought that he was with the party; but as it developed later he happened to be en route elsewhere. His obituary had been published, however. When the news reached Paso del Norte, his Mexican friends were most seriously disturbed and it was surprising how quickly the cavalry regiment was put in motion, the Colonel at its head. With Mr. Upham in danger, it was a matter of personal friendship rather than a routine sortie.

Following this, Mr. Upham very soon was offered and accepted the post of Resident and Chief Engineer of the Tampico Branch of the Mexican Central Railway, extending from Tampico to San Luis Potosi. This railroad he located and constructed. It was in very heavy work and very difficult location for which, no doubt, his experience in the Grand Canyon of the Arkansas especially qualified him. He was able to improve very materially on the location previously made by another engineer.

When this work was completed, he became, in 1884, Chief Engineer of the Colorado Railway Company, with headquarters at Denver, Colo. Shortly afterward he was appointed Chief Engineer of the Chicago, Burlington, and Northern Railroad Company, then under construction, with headquarters at St. Paul, Minn. Following this, he was appointed General Superintendent of all the lines in Illinois, of the Chicago, Burlington, and Quincy System. Temporary ill health resulting from exposure in the great strike on that System caused him to relinquish this post.

His health recovered, Mr. Upham was, in 1895, a Consulting Engineer, as well as Chief Engineer and General Manager of the Salt Lake Rapid Transit Company, operating a 30-mile electric line. Later, he was General Manager of a 60-mile electric street railway at Lincoln, Nebr., in which he also had some financial interest.

Somewhat later, Mr. Upham moved to New York, N. Y., where he became Consulting Engineer for the New York Steam Heating Company, and subsequently served as its Vice-President and General Manager; he remained in that position until ill health caused him to retire several years before his death which occurred in New York on January 7, 1928. While he was engaged with this Company he made an extensive study of methods of pulverizing coal and transporting it by water carriage through pipes from the mines to large cities. Nothing tangible resulted, however, whether because he was ahead of the times, or because of inherent difficulties in the process, it is impossible to say.

Mr. Upham was a member of the Sons of the American Revolution; the Society of Colonial Wars; and the Society of Mayflower Descendants.

He was a man of fine presence, tall, blonde, well built, physically and socially attractive, and of fine character; a gentleman in appearance, manner, and substance; he made many friends. As reflecting in some degree the influences which formed his character, his brother who compiled the Upham Genealogy which has supplied some of the foregoing data, in closing wrote:

> "If its effect is * * * as I have hoped it may be * * * to cause each one bearing the name of Upham to feel that he has responsibility for bearing it creditably, then indeed will a grand object have been attained, and we may adopt the sentiment of the New Brunswick Uphams:
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> "'If it is not in all mortals to command success, we will do more, deserve it.'"

Charles Clifton Upham qualified; he deserved success; he also secured it. The writer of this memoir, who was associated with him in Mexico and in New Mexico, holds him in fond remembrance.

He is survived by his widow, Anna St. John Eells, to whom he was married in December, 1883, and who also traced her ancestry to the early colonists of New England.

Mr. Upham was elected a Member of the American Society of Civil Engineers on April 7, 1897.

GEORGE SCHERZER WALSH, Sr., M. Am. Soc. C. E.*

Died May 28, 1929.

George Scherzer Walsh, Sr., was born at Lockport, N. Y., on December 22, 1871, the son of George M. and Ellen (Scherzer) Walsh. At the age of 8, his mother having died, he accompanied his father to Lincoln, Nebr., where the Walsh home was established.

Mr. Walsh was graduated from the Shattuck School, at Faribault, Minn., in 1890, and from the Rensselaer Polytechnic Institute, at Troy, N. Y., with the degree of Civil Engineer, in 1894. While at Rensselaer, he was affiliated with Delta Kappa Epsilon.

On the completion of his college course, Mr. Walsh went to San Salvador, Salvador, Central America, where he was associated until 1904 with his uncle, the late Albert J. Scherzer, M. Am. Soc. C. E., in the construction of the Santa Ana Railroad from the Port of Acajutla to Santa Ana, Salvador.

From 1904 to 1906, he was one of the Corps of Engineers on the Panama Canal construction, after which he went to Colombia, South America, and was in charge of the building of the Colombia Pacific Railroad, extending from Buena Ventura to Cali, Colombia. He was occupied in this work until September 28, 1909, when he returned to the United States.

In 1910, shortly after his marriage, he returned with Mrs. Walsh to Lincoln, where they lived until September, 1911, at which time Mr. Walsh was called back to Colombia as General Manager of the San Beningo Mines, a high placer operation on the Upper Porce River. He again returned to the United States in May, 1912, but went back to Colombia in November to continue the operation of the mine until May, 1913.

In January, 1914, Mr. Walsh sailed for London, England, to make certain reports to English capitalists on South American enterprises, and while abroad his son, George Scherzer Walsh, Jr., died in Lincoln. In February, 1915, he went to Panama, where he was Construction Engineer and General Manager for R. W. Hibbard and Company, on the David, Concepcion and Boqueti Railroad. This line opened up the mountainous region of Northern Panama. On the completion of this railroad, he returned to the United States just prior to its entrance into the World War.

Early in 1918, Mr. Walsh became interested in the Buffalo Brick Company, at Buffalo, Kans., the largest manufacturers of paving brick in the Middle West, and he went to Buffalo as General Manager of this Corporation. In June, 1918, he went into the Government service as District Engineer in charge of District No. 6 for the Emergency Fleet Corporation, with headquarters at Houston, Tex. After the Armistice he returned to Buffalo, where he again took charge of the Buffalo Brick Company on its re-opening in May, 1919. He operated the plant of this Company until his death, at Rochester, Minn., on May 28, 1929, where he had gone for treatment for heart trouble, which he contracted from influenza five years previous.

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* Memoir compiled from information on file at the Headquarters of the Society.

Mr. Walsh established an enviable record as a railroad and mining engineer in the United States and in Central America and South America. He was a devoted and true friend, wise in counsel, charitable, kind, and loyal. His ability, efficiency, integrity, and high standing among his associates commanded the respect and devotion of those who knew him both in professional and private life. He was a member of the Protestant Episcopal Church and of the Masonic Order.

He was married at Louisville, Ky., on June 1, 1910, to Mary Estes Leavell, of Lexington, Ky., who, with a daughter, Anna, and a son, Homan, survives him.

Mr. Walsh was elected a Junior of the American Society of Civil Engineers on February 2, 1897; an Associate Member on May 1, 1901; and a Member on March 31, 1908.

WILLIAM HENRY WARREN, M. Am. Soc. C. E.*

DIED JANUARY 9, 1926.

William Henry Warren, whose life was so intimately associated with the development of engineering education in Australia, was born in Bristol, England, on February 2, 1852, in the year that also saw the birth of the American Society of Civil Engineers, an institution for whose work and publications he always expressed the most profound admiration. It was, indeed, one of Professor Warren's striking characteristics that, although in private life a somewhat vehemently patriotic Britisher, yet, in matters affecting the Engineering Profession, he was a whole-souled Internationalist. To his classes he always spoke enthusiastically of the achievements of American engineers and especially of the great bridge builders, some of whom were his warm friends. He maintained a correspondence not only with several of the leading engineers of his own land, but also with many in the different countries of Europe, especially with men engaged in his favorite study—the investigation of the properties of materials of construction. Thus, to his mind, such a happening as the dissolution of the International Association for Testing Materials following as an indirect consequence of the World War could seem nothing less than a tragedy.

William Henry Warren was one of the numerous company of young Englishmen of his generation, who felt that in the Colonies of their Motherland would be found a wider field and ampler scope for their energies and ambitions. His early training as a Civil Engineer was the fairly customary one of the Sixties in England. In those days there were, of course, practically no engineering colleges as understood to-day. A lad obtained the elements of his professional knowledge under articles of indenture to some engineering office or works. Young Warren left school soon after he was 14 years of age and entered on a six years' pupilage in the London and North Western Railway Works, at Crewe, England. During this period he passed through the various departments of this famous Railway Company under the direction, first, of Mr. J. Ramsbottom and afterward of Mr. F. W. Webb, both of whom were among the best-known of the British railway men of those days.

He next spent three years in attending ordinary collegiate courses of study in pure and applied science in London and in Manchester, England, and achieved a worthy list of distinctions in the various examinations. Among other honors he was awarded a Royal Exhibition, a Whitworth Scholarship, and the Society of Arts First Prize and Scholarship. Then, possessing a good foundation both of scientific knowledge and workshop practice, Mr. Warren devoted six years to accumulating experience in the design and construction of various works in the north of England, the most important of them being the large Municipal Gas Works at Manchester. This enterprise was, for its date, a very notable undertaking, and Mr. Warren, who acted in the capacity of Designing and Supervising Engineer for the contracting firm, always looked on it as his most valuable piece of training and as concluding the "journeyman" period of his life, after which he was ready to devote the remainder to whatever might happen to be the ultimate task to which he was called. He little thought, at the time, that this task was to be chiefly the education of students for the Engineering Profession.

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* Memoir prepared by Sir Henry Barraclough, Dean of the Eng. Faculty, Univ. of Sydney, Sydney, N. S. W., Australia.

It was in 1881 that Mr. Warren finally left England and took up what was to be his permanent residence in Sydney, New South Wales, Australia. Here, he immediately obtained a position in the Public Works Department of the State and, at the same time, undertook in the evenings the direction of the Department of Mechanical Engineering at the Sydney Technical College. Toward the end of 1883 the appointment of Lecturer-in-Charge of the newly established Department of Engineering at the University of Sydney was offered him, and he thus began the long period of University service which closed only a few days before his death.

In view of Professor Warren's retirement from the Russell School of Engineering on New Year's Day, 1926, the Governing Council of the University of Sydney formally adopted the following resolution, which conveniently summarizes his life's work and reputation:

> "The Senate, having accepted with great regret, the resignation offered by Professor W. H. Warren, LL.D., of the Chair of Engineering, as from December 31st, 1925, hereby appoints him Emeritus Professor of Engineering from January 1st, 1926.
>
> "At the date of his retirement Professor Warren will have completed 43 years in the service of the University, 42 of them as Professor of Engineering, this being the longest occupancy of any Professorial Chair in the history of the University since its foundation. Included in this period were terms of office as Chairman of the Professorial Board, as Dean of the Faculty of Science, and as the First Dean of the recently established Faculty of Engineering. Professor Warren has been associated with the Department of Engineering from its earliest beginnings to its present recognised position as one of the leading Schools of Engineering in the British Empire. His devotion to the task of building up sound systems of Engineering teaching and research in Australia, and his enthusiasm in helping adequately to develop the Engineering Profession, have made his name a household word amongst the Engineers of the Commonwealth.
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> "Particularly in connection with the testing and investigation of the materials of construction, Professor Warren has greatly enhanced the reputation of the University. By his unremitting labours in this direction for nearly forty years the University now possesses a magnificent laboratory for the testing of materials, and by his industry and energy in carrying out researches into our natural supplies of building materials, Professor Warren has added greatly to the industrial resources of the country.
>
> "The Senate trusts that Professor Warren may live long to enjoy his leisure and to aid by his sympathy and advice the labours of those who follow after him."

This wish on the part of the Senate was not to be realized. Although arrangements had been made for Professor Warren to retain his old room at the University where he might do his technical writing—a plan that caused him much satisfaction—it was decreed that he should not return there again. The long vacation (of the Southern Hemisphere) had scarcely begun when the University was startled by his unexpected death. His end came suddenly and without pain or distress on January 9, 1926, while he was talking to a friend in his own home at Elizabeth Bay, Sydney.

To those who knew him intimately there was something fitting, something almost characteristic, in his end. Having finished his work he turned promptly to the next thing, as was his customary way. Promptness was to Professor Warren a cardinal virtue. Procrastination and unpunctuality were no failings of his. As his colleagues often had occasion to notice, when a letter on his desk was opened and read, his hand instinctively reached toward a sheet of paper and a pen to begin the reply—he wrote almost all documents by hand at the University, rarely dictating anything. The only hopeful way of tempting him on occasion to delay dealing instantly with a paper was to offer to draft the reply for him; he cheerfully and gratefully admitted the seductiveness of this method when the subject-matter permitted it. This plan of allowing one or more of his colleagues to draft a document for his consideration, especially where it affected the Engineering Faculty at large, became one of the recognized methods which, in his later years, was adopted by general consent.

Within these few lines no detailed account is possible of Professor Warren's professional achievements, of his services to the community on Royal Commissions of inquiry, and on the Councils of scientific and technical Societies, nor of his work in the Engineering Department of the University of Sydney. From his colleagues and brother engineers his labors received ample recognition. For several years he represented Australia on the Council of the Institution of Civil Engineers (England); he was elected as the first President of the Institution of Engineers, Australia, and twice was President of the Royal Society of New South Wales; the distinction that gratified him more than any other was the honorary degree of Doctor of Laws conferred on him by the University of Glasgow.

He was the author of many technical papers and reports, but was probably best known by his two-volume textbook "Engineering Construction, Part 1, In Steel and Timber; Part 2, In Masonry and Concrete," which has obtained a wide circulation. His investigation work was confined almost entirely to the study of construction materials. The equipment of the University Laboratory for the testing of materials was one of his chief preoccupations. Even if at times he was thought by some to be spending too large a proportion of available funds on equipment for this Laboratory, his efforts resulted in getting together a magnificent collection of testing machines and measuring apparatus culled from the standard types of the different nations.

During his forty-three years of service Professor Warren had seen his Department grow from two rooms, as its quarters, and himself as the entire Teaching Staff, to a large Faculty of Professors and Lecturers, and a range of laboratories that have few equals in the Engineering Schools of the British Empire. These things gave him immense and undisguised pleasure. Every advance made by the School (against what, especially in the earlier and middle years, he regarded as neglect and opposition) caused him to exult greatly. The Engineering School of the University was his chief hobby as well as his task.

Perhaps, indeed, it is not inaccurate to state that Professor Warren's attitude, generally, to life was the amiable one of the "hobby-ist". He had definite limitations, or, perhaps, more accurately, he deliberately limited his activities to those things in which he was naturally and keenly interested, and he was somewhat impatient of the others. One can rarely remember seeing him at work on anything for which he did not seem to have a zest. It was the same in his spare hours; as all his friends well know, he then devoted himself with equal enthusiasm to his three hobbies—golf, prize bulldogs, and music, especially singing. His knowledge of operatic music was a surprise to many professional singers. His remarkably trained voice and the obvious enjoyment with which he would let himself be persuaded to sing a series of songs, will long be a pleasant tradition.

His most striking quality was a very human one—people felt there was something about him "you could not help liking." It was this personal quality, perhaps, that gave him many friends, even among men with whom he had strong differences of opinion and active controversy, and that made him so popular with the graduates and undergraduates of the Engineering School. The student who first called him "Bill," and the occasion for the remark, are not recorded, but they belong to the very early days of the School. The name gathered an atmosphere of affection and respect with each succeeding generation of students, and to be so addressed as he always was on festive occasions, gave the "Professor of Engineering" genuine satisfaction.

It is proverbially difficult to merit one's epitaph, but Professor Warren was not entirely unworthy of the motto of his family crest inscribed on a memorial tablet erected to his memory in an English church where he was once a choir boy—*Cadenti porrigo dextram.* Especially among his old students are there men who realize that his cordial right hand did them many a kind turn.

For the earlier graduates in Engineering, for many men scattered in engineering positions over the surface of the globe, as well as for those who, like the writer, have long been his colleagues, it is difficult to think of the Engineering School in Sydney apart from Professor Warren. We are conscious of an element of astonishment at the idea of the School without him. We find it hard to imagine laboratories and lecture rooms in which his familiar figure does not come and go. It was, with us, in the Engineering School as if

"* * * having lived thus long, there seemed
No need the King should ever die."

The Russell School of Engineering with which Professor Warren was so intimately associated was setting off afresh the year he left it. New buildings were coming, additional laboratories were in project, and a larger staff called for. The School's long future will, doubtless, be greater in achievement than its past, but no other period can equal in romantic interest its first forty years, and no single individual will ever be so uniquely identified with its development as its first and (as it turned out) its only "Professor of Engineering."

Professor Warren was elected a Member of the American Society of Civil Engineers on February 5, 1890.

ROY IRVIN WEBBER, M. Am. Soc. C. E.*

Died May 14, 1929.

Roy Irvin Webber, the son of Irvin B. Webber, M.D., and Jane Mary (Wilson) Webber, was born in Warsaw, Ind., on August 27, 1876. He was enrolled in the course in Civil Engineering at Purdue University and was graduated in 1899. He obtained the degree of Civil Engineer from the University of Illinois in 1906, where he was an Instructor from 1902 to 1906.

Following his graduation from Purdue University, Mr. Webber worked for a year as Rodman for the Pennsylvania Railroad Company, and then spent six months as an Engineer for the Pressed Steel Car Company. From 1900 to 1902, he was Borough Engineer in Sewickley, Pa., from which position he went to the University of Illinois as Instructor in Civil Engineering.

In 1906, Mr. Webber was engaged as an Instructor in Civil Engineering at The Pennsylvania State College and was advanced to the rank of Associate Professor. In 1912, he was placed in charge of the Department of Architectural Engineering, a position which he held until 1918, when he was made Superintendent of Grounds and Buildings. In this last position, which he held at the time of his death, he had charge of the maintenance and operation of the college buildings and service plants, and either designed, or supervised the design of all new work and alterations.

At the time of Professor Webber's death, extensive building construction work was in progress for the College, and expansions had been planned for the immediate future with the $2 250 000 building fund appropriated by the State. All this work was under his direction and at the time of his death he was completing specifications for the architect on the remodeling of Old Main and the erection of a new power plant. He was in almost daily consultation with college officers concerning these buildings and had been working strenuously at his construction tasks for weeks, apparently in his usual excellent health. His death occurred very suddenly from a heart attack induced by acute indigestion. "It is indeed a great loss," said President Hetzel, of Penn State, "Mr. Webber was a most valuable man and one with whom it was a pleasure to be associated. We shall miss him."

Professor Webber was likewise actively interested in local affairs. He served as a member and President of the Borough Council and, in that capacity, he took part in the formulation of a comprehensive program of municipal improvements. He had also designed and constructed several business buildings and residences at State College. He was a Director of the First National Bank and a leading member of several social and fraternal clubs. He was an organizer and the first President of the State College Kiwanis Club, and ever since its organization he had been a leading member and officer in the State College Chamber of Commerce. He was also prominent in the Masonic fraternity, and a member of the Acacia fraternity.

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* Memoir prepared by P. B. Breneman, M. Am. Soc. C. E.

A resolution passed by the Board of Directors of the First National Bank of State College on the death of Professor Webber includes the following:

"*Resolved,* that we express our heartfelt and most solemn regret at the loss of one so worthy of the confidence and esteem of the community and who has contributed so much to the success of the several institutions with which he was identified. By his wise counsel and generous service, performed ofttimes at no little sacrifice of personal interest, he came to occupy an enviable place in the esteem of his co-workers and endeared himself to his official associates."

He was married to Olive Holtzberger, of Lafayette, Ind., who, with a son, Dean, and a daughter, Mrs. E. A. Dambly, survives him.

Professor Webber was elected an Associate Member of the American Society of Civil Engineers on April 1, 1908, and a Member on April 21, 1920.

TIMOTHY SIDNEY WHITE, M. Am. Soc. C. E.*

Died April 15, 1928.

Timothy Sidney White was born in New Brighton, Pa., on April 3, 1852. His parents, Timothy Balderston and Olive (Howland) White, were Quakers. He spent his early life in New Brighton, where he attended a local public school. From there he went to Cornell University, at Ithaca, N. Y., from which he obtained the degree of Bachelor of Science in Civil Engineering in the Class of 1873.

In 1835, or thereabouts, Mr. White's father had established a contracting business in New Brighton, specializing largely in the design and construction of masonry, timber, and steel bridges. This, it is thought, was the oldest company of its kind in Western Pennsylvania. It eventually grew into the Penn Bridge Company of Beaver Falls, Pa., which was incorporated in the early Eighties. Mr. White became Chief Engineer and eventually Vice-President and Chief Engineer, remaining in this dual capacity for about thirty-nine years. He retired from active business in 1924.

During this period the Bridge Company, under his engineering direction, grew rapidly. A large number of highway and railroad bridges throughout the eastern part of the United States were designed and built, all of which construction was done under Mr. White's direction as Chief Engineer. This work was highly diversified and included, not only bridges and buildings, but a great deal of specialized construction for the United States Government and various State, county, and city governments, and for many corporations as well. Among the important steel bridges built by the Penn Bridge Company are the four suspension bridges over the Ohio, at Rochester, East Liverpool, and Steubenville, Ohio.

Mr. White's professional life was identified with the development of the all-metal bridge from the small ones of wrought iron to the large modern bridges of high-tension steel. He was a student of engineering and deeply interested in the development of the steel bridge in general and especially in its details as they were related to facilities of manufacture. Under his direction as Chief Engineer, and that of his brother, Mr. S. P. White, as Business Executive, the Penn Bridge Company successfully undertook many important structures; some of them, such as the Anacostia Bridge and the Piney Branch Bridge in the District of Columbia, were largely masonry.

His work in structural engineering was nationally known, and he was highly regarded by the profession of bridge engineers throughout the country. He was a careful, conscientious, and most thorough engineer, and a student in the field to which he devoted his professional life.

Mr. White was a member of the Engineering Society of Western Pennsylvania, the American Society of Social and Political Science, and a Trustee and President of the Young Men's Christian Association at New Brighton. His clubs were the Duquesne Club of Pittsburgh, Pa., and Old Pueblo, of Tucson, Ariz.

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* Memoir prepared by Walter J. Douglas, M. Am. Soc. C. E.

He was an ardent Churchman, an active worker in the Protestant Episcopal Church, and prominent in the Diocese of Pittsburgh.

On June 8, 1876, he was married to Annie Augusta Appleton. He is survived by his widow and one son, Samuel A. White, of Beaver Falls, Pa.

Mr. White was elected a Member of the American Society of Civil Engineers on April 3, 1889.

WILLIAM HALSTED WILEY, M. Am. Soc. C. E.*

Died May 2, 1925.

William Halsted Wiley, the son of John and Elizabeth B. (Osgood) Wiley, was born on July 10, 1842, in New York, N. Y. In the early Fifties, Mr. Wiley's family moved to East Orange, N. J. Later, he attended the College of the City of New York until just prior to 1861 when the Civil War broke out. He received his Bachelor of Arts degree several years afterward.

He joined the Independent Corps of the New York Volunteer Light Infantry as First Lieutenant, an organization later known as the Seventh Regiment. Many years afterward, he was elected President of the Seventh Regiment War Veterans. In the course of the war, he rose to the command of two companies of artillery, seeing most of his service in South Carolina where he served with distinction in a number of severe engagements. He was given special mention by General Gillmore "for efficient and able services during the siege and bombardment of Fort Sumter and Charleston, S. C." In 1864, he was retired as Brevet Major "for gallant and meritorious services".

After the war, Major Wiley entered Rensselaer Polytechnic Institute and in 1866, was graduated from that institution with the degree of Civil Engineer, following this course with a year of special work at the Columbia School of Mines. For the next nine years he practiced engineering in the East and Middle West, with the Brooklyn, N. Y., Water-Works, the Croton Aqueduct, the Reading Railroad Company, the Riverside Park, of Chicago, Ill., and the Tunnell Hill Coal Company, near Zanesville, Ohio (as Superintendent). In 1876, he entered the publishing business with his father and his older brother, Charles, under the firm name of John Wiley and Sons.

The Wiley publications at that time included books of many sorts, but Major Wiley soon saw a great opportunity to develop the business into one of technical and scientific books. From his service in the war and as an engineer he noticed the great dearth of good American scientific literature. His friendship with officers of both the Army and Navy made it natural that he should be selected both by the West Point and the Annapolis Academies as the publisher of their books.

Major Wiley's wide and varied experience in the engineering field brought him into direct contact with the most prominent engineers of the country and gave him a first-hand knowledge of their needs. This knowledge enabled him to make many suggestions which led to the writing of needed engineering books by well-qualified authors. Thus, the business became definitely one of a scientific nature.

Under his able direction, which continued to the end of his life, the Wiley lists grew to include books in all the branches of engineering, agriculture, chemistry, geology, mathematics, and the biological sciences. In the last few years of his life, realizing the close link between engineering and business, he encouraged his associates to develop a group of books in the latter field.

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* Memoir prepared by Calvin W. Rice, Secy., Am. Soc. Mech. Engrs., New York, N. Y.

For many years he was the American correspondent of *Engineering* (London) and published through this paper an account of travels which he wrote with his daughter, Sara King Wiley, entitled "Yosemite, Alaska, and the Yellowstone".

Despite his great activity in the publishing field, Major Wiley found time for many other activities. For three years he was a member of the Township Committee of East Orange, N. J., serving for one year as its Chairman. He served three terms in Congress as a Representative from his district in New Jersey. He was a Commissioner from New Jersey to the St. Louis Exposition in 1904, and was President of one of the International Juries of the Brussels Exposition in 1897, being decorated with the Order of Leopold by the King of Belgium. He was a member of the Executive Committee of the National Security League and, during the World War, was Chairman of the National Preparedness Committee of the American Society of Mechanical Engineers. Major Wiley also was very active in the New York Commandery of the Loyal Legion.

He was a member of the American Society of Mining and Metallurgical Engineers, and the American Institute of Electrical Engineers. He was also a member of the American Society of Mechanical Engineers and served as its Treasurer for forty-one years. Major Wiley was one of the eighty men present at the meeting on March 15, 1880, at which the American Society of Mechanical Engineers was founded. He also belonged to The American Association for the Advancement of Science, and was a member of many clubs, including the University and Engineers' Clubs of New York, and the Cosmos Club, of Washington, D. C.

Major Wiley was one of the very early members of the Theta Xi Fraternity, being the third to be installed by the eight founders of the Fraternity and being in a large measure responsible for the development of its Constitution. He was its President for nearly two score years and was retired as President Emeritus only a few years before his death.

A Congregationalist, he was a member of the Trinity Congregational Church, of East Orange.

Major Wiley was married on June 1, 1870, to Joanna K. Clark, of Zanesville, Ohio. Both his daughter, Sara King, and Mrs. Wiley are deceased.

In all his engineering activities he was ever ready to lend a helping hand. His earnest efforts and foresight contributed much to the upbuilding of the National Societies and he stands out as an example of the type of pioneer who has given freely of his best efforts to make them what they are to-day.

The Major, as he was so kindly called, was most punctilious in all engagements while at the same time tempering his actions by a generous regard for the human relations. He was a past master at story telling. His sense of humor was always keen and, what is quite unique, he never repeated himself.

He was a man of close and strong friendships; a home lover. There was always that chivalry and consideration which bespeaks a true gentleman. Above all, he had the quality that convinced one of absolute truth and frankness which, with his kindly nature, endeared him to all. He was a man of large heart and keen and broad mind. His charity was without regard to position, creed, or color. For years, after the war, he found employment for men who had served under him. He was greatly loved and admired by his employees, as shown by the fact that many have remained in his employ for periods ranging from fifteen to forty-eight years.

In the death of Major Wiley, the Engineering Profession has lost not only an associate, but a benefactor. In the long period of his professional life, technical books have improved quite as much as has the science and practice of engineering. Major Wiley's name will go down in the annals of the profession, among the revered members, as one who served it well.

Major Wiley was elected a Member of the American Society of Civil Engineers on February 17, 1869, and thus, at the time of his death was one of its oldest members.

HENRY WILLIAM WILSON, M. Am. Soc. C. E.*

Died September 19, 1928.

Henry William Wilson, the son of William Hasell and Jane (Miller) Wilson, was born at Haverford, Pa., on August 25, 1843. He was tutored privately by Professor B. F. Green for two and one-half years, first at Glenmore and then at Beverwyck, between Albany and Troy, N. Y. He continued his education at Rensselaer Polytechnic Institute, at Troy, where he completed the course in two years and was graduated with the Class of 1864.

Mr. Wilson's professional career began when he entered the service of the Southern Pennsylvania Railroad Company, in 1865, as Rodman on surveys between the Allegheny Mountains and Chambersburg, Pa. In September of the same year he was advanced to the position of Second Assistant Engineer. In January, 1866, he was transferred to the West Pennsylvania Railroad, in charge of surveys in Butler and Allegheny Counties, Pennsylvania, and, in February, was appointed First Assistant Engineer.

During this period, Mr. Wilson was in the office of the Principal Assistant Engineer at Altoona, Pa., which office was transferred to Philadelphia, Pa., in December, 1867. He was also engaged in various kinds of field work, surveying, locating, and superintending construction, etc., all of which was done principally under the direction of his father, the late W. Hasell Wilson, Hon. M. Am. Soc. C. E., who was Chief Engineer. His work in this capacity also included the revision of calculations and plans for the Pennsylvania Railroad Bridge over the Delaware River, at Trenton, N. J., for which he later became Superintendent of Construction.

In 1876, Mr. Wilson entered the office of Wilson Brothers and Company, Engineers and Architects, of Philadelphia, where he was engaged on the design of bridges and other construction work for the firm, under the direction of the late J. M. Wilson, M. Am. Soc. C. E., Engineer of Bridges and Buildings for the Pennsylvania Railroad Company. Mr. Wilson made annual inspections of the bridges of the Railroad System with the Engineer of Bridges and Buildings, who submitted reports to the General Manager. These reports included recommendations as to conditions and the necessary work to be done in bridge construction.

Subsequent to 1885, when the late Mr. J. M. Wilson left the service of the Pennsylvania Railroad Company, Mr. Wilson, as a member of the firm of Wilson Brothers (later, Wilson, Harris, and Richards), attended to all the engineering and structural work until his retirement from active business in 1910. He designed the structural work for the Drexel Building and for the Edison Electric Light Building at Ninth and Sansome Streets, Philadelphia, and made calculations for the bridges on the Atlantic Coast Line Railroad, submitting reports as to their condition and recommending the necessary renewals. He also made examinations and calculations of bridges for the Buffalo, Rochester, and Pittsburgh Railroad Company.

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* Memoir prepared from information on file at the Headquarters of the Society.

Mr. Wilson was a member of the Franklin Society. On October 15, 1874, he was married to Harriet McFarlan Morton, the daughter of the Rev. Henry J. Morton, of Philadelphia. They had one son, Arthur Morton Wilson, who resides in Philadelphia.

Mr. Wilson was elected a Member of the American Society of Civil Engineers on September 6, 1876.

IRVING MASON WOLVERTON, M. Am. Soc. C. E.*

Died January 6, 1930.

Irving Mason Wolverton was born at Grand Blanc, Mich., on January 29, 1869, and, in 1880, moved with his parents to Flint, Mich. After graduating from the High School at Flint, he entered the Civil Engineering Department of the University of Michigan, from which he was graduated in 1890.

After graduation Mr. Wolverton became associated with the King Bridge Company of Cleveland, Ohio, as an Engineer on structural steel work. After four years with that Company, he was appointed Chief Engineer of The New Columbus Bridge Company, of Columbus, Ohio, with which he remained until 1899.

He was then chosen Chief Engineer of The Mount Vernon Bridge Company of Mount Vernon, Ohio. He successively was made Vice-President and, in 1919, President and Treasurer, which latter position he held until his death. The Mount Vernon Bridge Company has built many notable highway and railroad structures in various parts of the United States, and, under the guidance of Mr. Wolverton, has become one of the leaders in the field of steel construction.

Death came to Mr. Wolverton very suddenly on January 6, 1930. He had gone to Columbus on a business trip and stayed over Sunday in order that he might see a physician who was to be out of town until Monday. It was thought that his trouble was only a cold. Mrs. Wolverton and several of his associates saw him on Sunday, and he seemed to be in very good condition. On Monday morning he was found dead in his bed at the hotel.

Mr. Wolverton had a reputation for loyalty to his friends and for doing well everything that he attempted. He went into all propositions carefully and in detail. His success was due to this painstaking care and to his ability to make and keep a host of friends. He was a member of the Mount Vernon Country Club and The Columbus Club, of Columbus. He was also a member of the Benevolent and Protective Order of Elks and was a Thirty-second Degree Mason.

He was married on September 12, 1893, to Florence Harriet Pope, of Allegan, Mich., and is survived by his widow, three daughters, Carlotta Schafer, of Cleveland, Ohio, Harriet Schelling, of New York, N. Y., and Frances Taylor, of Middletown, Conn., and one son, John P. Wolverton, of Akron, Ohio.

Mr. Wolverton was elected an Associate Member of the American Society of Civil Engineers on December 4, 1895, and a Member on December 1, 1903.

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* Memoir prepared by C. G. Conley, M. Am. Soc. C. E.

IRVING WORTHINGTON, M. Am. Soc. C. E.*

Died May 27, 1928.

Irving Worthington was born at Sauk Center, Minn., on June 19, 1868. His mother was Sarah Lewis, of New York State, and his father, Leslie Worthington, was descended from a Connecticut family which claimed a Revolutionary War ancestry on his mother's side. He was related to the late Henry Worthington, M. Am. Soc. C. E., of New York, N. Y.

Irving Worthington's education was begun in the schools of Sauk Center. Later, when the family moved to Barrie, N. Dak., he finished his common school education there and also taught for one year. He moved to Spokane, Wash., in 1889, and was enrolled in the pre-engineering course at Spokane College, where he was a student for two years.

Mr. Worthington's first engineering work in 1891 was on Government surveys under Mr. J. K. Ashley, of Spokane. From 1893 to 1903, he was engaged in the private practice of general engineering and maintained an office in Spokane. During this period, he surveyed much of Chelan and Okanogan Counties, Washington and, as Deputy Mineral Surveyor, the mineral belts and mines of Washington, Oregon, Idaho, and Montana.

In 1903, Mr. Worthington began his long experience in the irrigation field under Ernest McCulloh, M. Am. Soc. C. E., for the Yakima Development Company, of Yakima, Wash. He was active in irrigation investigations for this Company for three years. From 1905 to 1908 he was in charge of important irrigation developments for the Oregon Land and Water Company. During the next five years he was Chief Engineer of the Rogue River Valley Canal Company, Medford, Ore. This project embraced more than 50 000 acres, and included two storage reservoirs, and many miles of canal and laterals, as well as the problems of land settlement. The writer had knowledge of Mr. Worthington's work on this project from personal observation and the results of his labor are now apparent in the productiveness under irrigation in Rogue River Valley.

In 1913, he was called to be Assistant Engineer and Water Superintendent for the Fresno, Calif., Canal and Irrigation Company and continued as such until his appointment as Engineer Appraiser for the Federal Land Bank, at Spokane, which post he held until his death on May 27, 1928.

Mr. Worthington was married to Frances C. Brattain, of Spokane, in March, 1900. Mrs. Worthington, with their three children, Patricia, Hugh, and Jean, survives him.

He was a likeable man, a conscientious and painstaking engineer. Whatever work he undertook, was well done.

Mr. Worthington was elected a Member of the American Society of Civil Engineers on June 24, 1914.

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* Memoir prepared by William J. Roberts, M. Am. Soc. C. E.

ROBERT HARLOW ANDERSON, Assoc. M. Am. Soc. C. E.*

Died August 8, 1929.

Robert Harlow Anderson was born in North Woburn, Mass., on October 2, 1877. He was educated at Robert College, Constantinople, Turkey, at Williams College, Williamstown, Mass., and at Cornell University, Ithaca, N. Y., receiving the degree of Civil Engineer from the latter institution in 1898.

In June, 1898, Mr. Anderson began his professional career as Rodman and Draftsman on the United States Deep Waterway Survey through New York State. From December to March, 1900, he served as Draftsman for James H. Fuertes, M. Am. Soc. C. E., on an investigation of the city water supply for the Merchants' Association of New York City. In March, 1900, he was engaged as Draftsman and Transitman on surveys for the Nicaragua Canal and, in 1901, became Transitman for the late George S. Morison, Past-President, Am. Soc. C. E., on water power investigations on the Susquehanna River. From 1903 to 1904 he was with the United States Geological Survey and in conjunction with John C. Hoyt, M. Am. Soc. C. E., prepared a report on the hydrography of this river.† After the completion of this report he was appointed Assistant Engineer with the Isthmian Canal Commission. Mr. Anderson was one of the first engineers on the Panama Canal. He was engaged in making investigations and plans, under the Resident Engineer at Bas Obispo, Panama, for the control of the Chagres River.

From 1905 to 1909 he served as Engineer for the McCall's Ferry Power Company on the Holtwood, Pa., Power Dam, and he was also employed on work in connection with an irrigation project at Twin Falls, Idaho, and for a hydro-electric project in Nova Scotia. Mr. Anderson's first connection with the Tennessee Electric Power Company was in 1909 when, as Resident Engineer for the J. G. White Construction Company, he was engaged on the construction of two hydro-electric developments for the Power Company at Parksville and Caney Creek, on the Ocoee River. From 1915 to 1917 he was located at Rock Island, Tenn., in charge of the construction of the Great Falls Hydro-Electric Development for the same Company, and, from 1919 to 1921, he was Engineer for the J. G. White Construction Company, located at Rifton, N. Y., during the building of the dam and power plant on the Wallkill River. He also was engaged on other engineering work for the J. G. White Construction Company at Langley Field, Virginia, and at the U. S. Nitrate Plant at Muscle Shoals, Alabama.

In 1922, Mr. Anderson returned to The Tennessee Electric Power Company as Hydraulic Engineer, which position he held at the time of his death. During 1923 and 1924 he was in charge of the construction of additions to the dam and power-house at the Great Falls Plant. Subsequently, he made surveys and engineering investigations of various projects and was in charge of the construction of an earth storage dam on the Toccoa River, for the Toccoa

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* Memoir prepared by J. A. Longley, Esq., Chattanooga, Tenn.

† *Bulletin No. 109*, U. S. Geological Survey.

Electric Power Company, a subsidiary of The Tennessee Electric Power Company. This dam is to be 167 ft. high and will contain approximately 1 600 000 cu. ft. of earth, with a power installation of 25 000 h.p. Mr. Anderson was actively in charge of this work until his death.

Mr. Anderson is survived by his widow, Mrs. Elizabeth Fry Anderson, of Sunnyburn, Pa., to whom he was married in 1907. He also leaves one brother, Roger H. Anderson, an Attorney of New York City, and two sisters, Mrs. George Baker, of Altadena, Calif., and Sarah Anderson, of Pasadena, Calif. His father, Charles Anderson, who died in 1928, was Dean of Robert College, Constantinople, until his retirement in 1917, when he returned to the United States.

Mr. Anderson leaves a host of personal friends, both within and without The Tennessee Electric Power Company, who held him in deep admiration for his sterling qualities as a man and a most capable engineer.

Mr. Anderson was elected an Associate Member of the American Society of Civil Engineers on January 8, 1908.

BERTRAND DON BARKER, Assoc. M. Am. Soc. C. E.*

Died November 7, 1929.

Bertrand Don Barker, the son of Don Barker and Adelia Barker, was born on June 17, 1881, at Poultney, Vt. His father was engaged in the operation of State quarries. The boy grew up in New England, attending the local schools. In 1903, he received the degree of Bachelor of Science in Civil Engineering from Norwich University, at Northfield, Vt.

Immediately after his graduation, Mr. Barker entered the service of the United States Coast and Geodetic Survey where his ability and energy brought him advancement. By July, 1905, he was Executive Officer of the U. S. S. *Hydrographer.* He served in this capacity on the *Hydrographer* and the *McArthur* until January, 1907, and in other capacities until May, 1907, when he resigned from the Service.

Mr. Barker next became connected with George W. Jackson, Incorporated, of Chicago, Ill., and New York, N. Y., as Engineer and General Foreman and Superintendent of tunnel construction. From January to August, 1910, he was Superintendent of Shaft No. 1 of the Washington Street Tunnel, Chicago, and from August, 1910, to February, 1912, General Superintendent of Shaft No. 1 of the Yonkers Pressure Tunnel on the Catskill Aqueduct.

In June, 1912, Mr. Barker became associated with George A. Quinlan, M. Am. Soc. C. E., Contracting Engineer of Chicago. Early in 1914, Mr. Quinlan was appointed County Superintendent of Highways of Cook County, Illinois, under the newly enacted Road Law. On assuming this position he took Mr. Barker with him as Chief Engineer.

When the United States entered the World War, Mr. Quinlan went into the Army, and Mr. Barker assumed temporary charge of the County Highway Department. In this capacity, throughout the period of the war, he rendered invaluable, if unrecognized service, to his country by developing and maintaining highway transportation facilities in the great industrial district about Chicago. On the return of Major Quinlan from the Army Mr. Barker was again able to devote his entire time to the engineering work of the Department. With the impetus given to road building by the war, and with funds available from both County and State Highway bond issues, Mr. Barker found plenty to do.

In 1925, with three of his friends, he organized the firm of Barker, Flavin, Sheets, and Wallace, Incorporated, Engineers, with offices in Chicago. Such time as he could spare from his public duties, he devoted to this firm of which he was President at the time of his death.

He was married to Mabel Gibbs, on January 18, 1905, at Chicago. He is survived by his widow and only daughter, Mary Ciel, and by his sisters, Mrs. Alice Jones and Mrs. Allyne Walker.

Despite the strenuous activities of his County Highway Office Mr. Barker always took an active interest in the affairs of the Society. Like many other men who are busy he postponed filing application for transfer to the grade of Member, although the nature of his work and the part he took in it would undoubtedly have entitled him to this honor.

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* Memoir prepared by C. C. Wiley, M. Am. Soc. C. E.

As an engineer, Mr. Barker was capable, far-seeing, resourceful, energetic, and skillful. The development of the highway system in the metropolitan area around Chicago owes much to him. As a man, he possessed a brusque but delightful personality. His wide interest in things, his keen mind, and his ready sense of humor, made him liked by all, and he is mourned by his many friends and associates.

On the night of November 7, 1929, Mr. Barker was killed while on his way home from a meeting relating to highway affairs in one of the suburban villages, being struck by a switch engine at the crossing of the Indiana Harbor Belt Railway and West 74th Street, Chicago.

He was particularly active in securing separation of grades between railroads and highways wherever possible in the County. At the time of his death he was presenting the County's case before the State Utilities Commission for separation of grade on the particular crossing at which he was killed.

Mr. Barker was elected an Associate Member of the American Society of Civil Engineers on September 3, 1913.

JOHN CLARK BENTLEY, Assoc. M. Am. Soc. C. E.*

Died September 24, 1928.

John Clark Bentley was born at Westerly, R. I., on November 17, 1881, the son of Benjamin Courtland Bentley and Henrietta (Clark) Bentley. His father was a leading citizen and prominent builder of Westerly. The boy's preparatory school education was obtained in the public schools of his native town, and he was graduated from Rensselaer Polytechnic Institute at Troy, N. Y., with the degree of Civil Engineer in 1904.

During his summer vacations, Mr. Bentley supplemented his studies by gaining practical experience as Chainman and Transitman on surveys at Plainfield, N. J., and at Westerly. After his graduation he entered the employ of the New York, New Haven, and Hartford Railroad Company, and as Chainman and, subsequently, as Transitman, was engaged on surveys, plans, and estimates, and supervising the removal of bridges and the completion of the elimination of grade crossing at Webster Avenue, New Rochelle, N. Y.

In November, 1905, Mr. Bentley entered the service of the New York, Ontario, and Western Railway Company, at Middletown, N. Y., as Assistant Engineer in charge of preparing plans for maintenance and construction and also of engineering on 240 miles of track, under the Engineer, Maintenance of Way. Among the different projects that he planned and supervised during this time were a retail coal storage plant at Boston, Mass., several bridges, a 980-ft. steel viaduct, the reconstruction of a coal shipping pier at Weehawken, N. J., various grade crossing eliminations, station and shop buildings, yards, etc.

Following a period of several years' service with the Railway Company, and until the latter part of 1917, he was engaged independently in various construction contracts at Elizabeth, N. J., on State road contracts in Maryland, in association with Mr. E. C. Humphry, of Hackensack, N. J., and, again, individually on road contracts at Lakewood and Plainfield, N. J.

After the completion of his road contracts with the City of Plainfield, in the latter part of 1917, Mr. Bentley disbanded his own organization and accepted a position as Superintendent of Road Construction with Snare and Triest, Contractors, on the Raritan Arsenal at Bonhamtown, near New Brunswick, N. J., where he remained until the middle of 1919. During this period, which embraced the extremely cold weather of 1917-18, he forced to completion at record speed, the construction of a large system of main concrete roads and subsidiary gravel roads throughout the Arsenal and also handled the construction of a number of the permanent masonry Administration Buildings.

In 1920, Mr. Bentley decided to re-enter the general construction business on his own account and formed a partnership with Mr. Joseph A. Morrison, of Plainfield, operating under the name of Bentley-Morrison, Engineers and Constructors, with headquarters at Elizabeth. In 1922, this partnership was dissolved and the Bentley-Morrison Corporation was incorporated to carry on the business of the partnership. Mr. Bentley was made President of the Corporation, which position he held until April, 1927, at which time he disposed of his interest to the other stock holders. During this period, from 1920 to 1927, his organization was engaged on a large variety of contracts in the building and railroad construction field, with such concerns as the Willys Corporation, the Durant Motor Company, the Singer Manufacturing Company, the Standard Oil Company of New Jersey, the Reading Company, the Pennsylvania Railroad Company, the Lehigh Valley Railroad Company, and others.

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* Memoir prepared by Joseph A. Morrison, Esq., Plainfield, N. J.

Mr. Bentley's disposal of his interest in the Corporation of which he was President, was probably somewhat influenced by the condition of his health, which since 1925 had caused him no little concern. He suffered a nervous breakdown in 1926, but after a long vacation in the Adirondacks he returned to the management of his affairs considerably improved in health. In the summer of 1927, he resumed business, as an individual, at Elizabeth, being engaged in building and real estate projects until his sudden death on September 24, 1928.

Mr. Bentley was a man with considerable force of character who brought a keenness of mind and enthusiastic purpose to bear upon the solution of the many problems that presented themselves in the conduct of the affairs with which he was engaged during an active career. He was a member of the Plainfield Lodge of Benevolent and Protective Order of Elks.

He was married, in 1905, to Charlotte Perry, of Troy, whose decease occurred shortly before his own. He is survived by his mother, Mrs. Henrietta Clark Bentley, and one sister, Bertha Bentley, both of Plainfield.

Mr. Bentley was elected an Associate Member of the American Society of Civil Engineers on May 31, 1910.

ERNEST ARDEN BRUCE, Assoc. M. Am. Soc. C. E.*

Died March 8, 1928.

Ernest Arden Bruce, the son of J. A. and E. Lee (Wiseman) Bruce, was born at Ingleside, W. Va., on January 28, 1887. He spent his early life in Bluefield, W. Va., where he fitted himself for college. He entered Concord College in 1904; later, he became a student in West Virginia University from which institution he was graduated in 1908 with the degree of Bachelor of Science in Civil Engineering.

Mr. Bruce began his professional career in Bluefield, where he served as Transitman, and, later, as City Engineer. He also served in the capacity of City Engineer for the Cities of Princeton and Beckley, W. Va. From 1915 to 1917 he was engaged as Assistant City Engineer of Charleston, W. Va., and from 1919 to 1923, he was City Engineer of Charleston, after which he resumed his private practice as Civil and Sanitary Engineer.

He organized the Bruce Construction Company, which Company became extensively engaged in the general contracting business of building roads, streets, and sewers. Mr. Bruce was a member of this Company at the time of his death.

He was a member of the American Association of Engineers, and President of the State Board of Registration of Engineers of West Virginia. He was an engineer of unusual professional attainments, ever active in promoting the increased recognition of Registered Professional Engineers by the public.

Mr. Bruce ably served on many engineering committees appointed to co-operate with various civic bodies in the handling of local public problems. He was considered one of the outstanding leaders in his profession, whose judgment was sound on technical matters.

He possessed a lovable disposition which endeared him to all who were privileged to know him; his untimely passing spread universal sorrow among his friends, and was a distinct loss to his profession.

Mr. Bruce was elected an Associate Member of the American Society of Civil Engineers on April 14, 1919.

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* Memoir prepared by Philip Joseph Walsh, Assoc. M. Am. Soc. C. E.

THOMAS HIBBEN CLAUSSEN, Assoc. M. Am. Soc. C. E.*

Died November 9, 1928.

Thomas Hibben Claussen, the son of William Dudley and Mary (Hibben) Claussen, was born at Sausalito, Calif., on March 26, 1892. His father was born in Algiers, Africa, and was raised a British subject, whereas his mother was of Canadian birth. Mr. Claussen's boyhood life was spent at Sausalito where his mother still resides. His father died in 1920.

While attending the public schools in his native city Mr. Claussen at one time was a pupil of Kathleen Norris, the novelist. At an early age he entered the Hitchcock Military Academy, at San Rafael, Calif., from which he was graduated in 1910. From 1910 to 1916 he studied civil engineering at Leland Stanford Jr. University at Palo Alto, Calif., with brief intermissions during which time he was in the employ of the City of Sausalito as Draftsman on street work, sewer plans, and construction work. Early in 1917, he entered the employ of the Spring Valley Water Company, of San Francisco, Calif., as Draftsman on general water supply plans.

In September, 1917, Mr. Claussen enlisted in the military service and soon attained the rank of Sergeant in the Intelligence Section of the 363d Infantry. After a period of about one year in this branch, he was commissioned Second Lieutenant of the 50th Field Artillery, 91st Division, and acted as Instructor, training recruits in the military camps in the Eastern and Southeastern parts of the country until the close of the World War. He was honorably discharged from the service on December 26, 1918.

During the first half of 1919 Mr. Claussen served two engagements of about three months each, with the Gorman Engineering Company and Hunter and Hudson, Consulting Engineers, of San Francisco.

August, 1919, brought an important change to Mr. Claussen when he entered the United States Reclamation Service where, for the next six years, he filled several responsible assignments, the first of which was as Assistant Engineer in charge of silt survey, at the Elephant Butte Reservoir. He then, successively, acted as Assistant Engineer and Chief Hydrographer on the Rio Grande Project; made studies of water surface curve, standing wave, and upward pressure, at the Percha Weir; investigated the consumptive use of water on the Rio Grande Project; determined stresses at the Elephant Butte Dam; and studied the effect of back-water from the Elephant Butte Reservoir.

In October, 1922, as Assistant Engineer, he aided in gathering data for a report on flood control for the Lower Rio Grande Valley; subsequently, he assisted on a report on the irrigation of Hope Community from Penasco River, including a reconnaissance survey and office compilations; and assisted on a report for a tri-country project in Nebraska. He was transferred to the Yuma Project in November, 1923, where he held a position as Assistant Engineer until July, 1925, when he retired from the Reclamation Service to enter the service of the State of California.

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* Memoir prepared by M. S. Edson, Assoc. M. Am. Soc. C. E.

On August 9, 1925, Mr. Claussen accepted a position with the Division of Engineering and Irrigation of the State of California which Department was then engaged in a comprehensive State-wide water conservation survey. He remained with this Department until February, 1927, when he was forced to give up his position, due to ill health.

On March 7, 1924, he was married to Winifred Dodds, of Beatrice, Nebr., who, with his mother, survives him.

During his short career Mr. Claussen won the admiration of a host of friends due to his unfailing good nature and staunchness of character, and his early demise is keenly felt by all who knew him.

Mr. Claussen was elected a Junior of the American Society of Civil Engineers on November 25, 1919, and an Associate Member on May 19, 1924.

ADOLPHUS JAMES EDDY, Assoc. M. Am. Soc. C. E.*

Died June 28, 1929.

Adolphus James Eddy, invariably known as "Jim" Eddy, the son of Adolphus Frederick and Mary Ellen (Slover) Eddy, was born at Jacksonville, Ore., on December 25, 1885. He completed his Grammar and High School education in Southern Oregon and moved to San Francisco, Calif., in 1904, where he worked for two years in the shops of the Union Iron Works, earning sufficient money to start his college course.

In 1906, Mr. Eddy entered the University of California and in May, 1910, was graduated with honors in Civil Engineering, Astronomy, and Military Science. He was one of the representatives of the Class selected to speak at the Commencement Exercises. For the following two years, he carried on post-graduate work at the University in Civil Engineering and allied subjects.

He began his professional career as early as 1902, when for eleven months he worked as Chainman and Instrumentman for the Southern Pacific Railroad Company. In June and July, 1908, he acted as Inspector on the construction of a highway in San Mateo County, California. After graduation from college in 1910, until 1917, and again in the early part of 1920, Mr. Eddy was on the Teaching Staff of the College of Civil Engineering at the University of California, as Instructor, Assistant Professor, and, finally, Associate Professor. During this period, he also served as Assistant Instructor in Military Science.

During the same period, Mr. Eddy devoted all his spare time to professional work, including structural designing for John Galen Howard in the summers of 1910 and 1915; for Frederick H. Meyer in 1911; for the Panama-Pacific Exposition Company in 1914; and for Charles Derleth, Jr., M. Am. Soc. C. E., in 1916.

In the fall of 1919, following his military service in the World War, Mr. Eddy was employed for three months by Fred H. Tibbetts, M. Am. Soc. C. E., on field surveys and office investigations of an irrigation project in the Sacramento Valley, involving about 5 600 acres and requiring provision for flood protection and drainage, as well as irrigation service. Commencing in May, 1920, he was employed for about three years by the Standard Oil Company of California on structural engineering work, pertaining largely to the design and construction of the Company's 22-story office building, which was one of the first high buildings constructed on or near the filled-in area of San Francisco. This work involved important and thorough investigations of foundations and complete checking of the steel frame design, special attention being given to possible earthquake effects.

In September, 1923, shortly after the inauguration of the City Manager form of government in Berkeley, Calif., Mr. Eddy was appointed City Engineer and Superintendent of Streets for that city. His work involved a wide variety of engineering problems, including street paving, sewerage systems, storm drainage, garbage disposal, concrete retaining walls, and other structures. Studies and reports were also made on Berkeley's municipal wharf, including plans for extensive repairs and additions, and preliminary studies were made for a municipal airport.

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* Memoir prepared by the following Committee of the San Francisco Section, Ralph G. Wadsworth, *Chairman*, and C. A. Whitton, Members, Am. Soc. C. E., and E. A. Reinke, Assoc. M. Am. Soc. C. E.

Mr. Eddy always took an extremely active interest in military work of various kinds, beginning with a two-year period of service in the Oregon National Guard in 1902-1903. At the University of California, he completed all the courses in Military Science and was graduated with the commission of Major in the Cadet Corps. Prior to the entrance of the United States into the World War, he was active in organization of citizen-soldier activities at the University, and in the spring of 1917 he entered the first Officers Training Camp at the Presidio of San Francisco, from which he was graduated with a commission as First Lieutenant, Coast Artillery. On November 30, 1917, he was promoted to the rank of Captain, and on November 9, 1918, to that of Major. He took a command of 300 enlisted men and 4 officers, comprising an Army Artillery Park, to France and, while there, attended the Heavy Artillery School at Angers. At the time of the signing of the Armistice, he had just been ordered back to the United States in order to conduct overseas another artillery unit. In January, 1919, at Camp Eustis, Virginia, he had full charge of the test firing of an 8-in. howitzer on a self-propelled mount, and, later in that year, conducted a recruiting party through the Middle Western and Pacific Coast States. He was given an honorable discharge on November 6, 1919.

On May 4, 1921, he was appointed Captain of the 159th Infantry, California National Guard, and was rapidly advanced through the several intervening grades to the rank of Colonel, commanding the regiment, which rank he held from June 29, 1925, until his death. At the time of the disastrous fire, which swept through thirty blocks of high-class residences in Berkeley, in 1923, he risked official censure by ordering out, on his own responsibility, the local National Guard unit, and earned the gratitude of the community for the effective police and relief work accomplished under his direction.

His death, on June 28, 1929, occurred as the direct result of a major operation performed in San Francisco to cure an intestinal disorder, which had given him intermittent trouble subsequent to his service in the war.

Mr. Eddy was a member of Berkeley Lodge No. 363, F. and A. M., the Oakland Scottish Rite Bodies, and Aahmes Temple, A A. O. N. M. S. He was a Director of the Berkeley Chamber of Commerce, a member of the Rotary Club, the Faculty Club of the University of California, the American Legion, the Association of the Army of the United States, and various other societies. His scholastic honors included membership in Tau Beta Pi, Phi Beta Kappa, and Sigma Xi. His sterling character, charming personality, and boundless good humor won him the friendship and admiration of all who knew him.

He was married at Berkeley, in August, 1916, to Margaret G. Stone. They had three children, Margaret Eleanor, James Stone, and Barbara, all of whom, together with his father and mother, survive him.

Mr. Eddy was elected a Junior of the American Society of Civil Engineers on December 6, 1910, and an Associate Member on July 6, 1920.

PAUL MAX ENTENMAN, Assoc. M. Am. Soc. C. E.*

Died March 29, 1929.

Paul Max Entenman, the son of Herman and Marie Entenman, was born in New York, N. Y., on January 19, 1887. After graduating from Public School No. 77 in Manhattan in 1900, he was a student at the Morris High School in The Bronx for two years. He attended engineering courses at Cooper Union from 1902 to 1907, and was graduated as the Valedictorian of his Class, with the degree of Bachelor of Science. Among his classmates was Morgan F. Larson, Assoc. M. Am. Soc. C. E., the present Governor of New Jersey. Mr. Entenman continued his studies by taking evening courses at the Polytechnic Institute of Brooklyn from 1907 to 1909, from which he was graduated with the degree of Civil Engineer. Later, he did some post-graduate work in special subjects at Columbia University and at the Brooklyn Institute of Arts and Sciences.

From August, 1904, to June, 1906, Mr. Entenman served as a Rodman and Topographical Draftsman with the Rapid Transit Railroad Commission, New York City, on the construction of subways. He was promoted to the grade of Assistant Engineer, holding that position under the same Commission and its successor, the Public Service Commission for the First District, until November, 1914. He had wide experience in the various types of subway construction, including deep tunnels in rock, cut-and-cover work in rock and in soft ground, and in the underpinning of buildings as well as in the many complicated features incidental to such construction.

He was very much interested in and applied himself zealously to this work, approaching all the problems with a true engineer's mind, taking nothing for granted, but searching out the reasons. He was indefatigable in his attention to his duties. During this time he made the acquaintance of the late Clifford M. Holland, M. Am. Soc. C. E., then a young Assistant Engineer on the same work, who, later, became the Chief Engineer of the Vehicular Tunnel under the Hudson River, which bears his name. The two men became firm friends and this inspiring and unselfish friendship endured until death separated them. Each had a high respect for the other's character and attainments, and each was generous in his praise of the other's accomplishments.

Mr. Entenman's strenuous devotion to his work overtaxed him and his health suffered in consequence, so that in November, 1914, he gave up his position in New York and went to Los Angeles, Calif., where he spent about two years recuperating. He then decided not to return to the East, but to take up work in Southern California.

In October, 1916, he went to Calexico, Calif., on the Mexican border, to work with Joseph C. Allison, M. Am. Soc. C. E., and in April, 1919, they formed a partnership for the general practice of engineering with offices at Calexico and Los Angeles. This partnership continued until March, 1923. Among other activities, Mr. Entenman was engaged on irrigation work in the Imperial Valley, on flood control and irrigation on the Colorado Delta in

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* Memoir prepared by Robert Ridgway, Past-President, Am. Soc. C. E.

Lower California, and as Engineer of the Palo Verde Irrigation District at Blythe, Calif. A system of levees was designed and built to protect the Palo Verde Basin from the floods of the Colorado River. From 1919 to 1921 he also served as the City Engineer of Calexico, which then had a population of about 10 000. During this time he designed and built its water-works system as well as other public improvements.

A relapse in 1921 led Mr. Entenman to remove to Banning in the San Gorgonia Pass on the road from Los Angeles to Imperial Valley, where he built the home in which he lived until his death. Although his illness prevented him from traveling about and doing field work he remained active otherwise. From his home he directed in 1923, a ground-water survey in Lower California and Mexico, for the Colorado Land Company, to ascertain the feasibility of obtaining, by pumping, a supply of water for irrigating land too high to be reached by gravity from the Colorado River. In 1926, he designed for Mr. Allison the Bee River Siphon in Lower California. This was a true siphon, consisting of pipes designed to take water from the Colorado River over the levee into the Bee River—an old channel once occupied by the Colorado River —to irrigate delta lands to the west. The siphon was completed in 1928, and has operated very successfully. The design required a careful research into engineering literature relating to the principles of true siphons. It was Mr. Entenman's intention to prepare a paper for the Society describing the unique features of this work, but death intervened before he completed it.

His experience with works in the Colorado River Basin led him to make a broad and careful study of the problems and characteristics of that river and he contributed to the discussions concerning them in the Society's *Transactions* and in the technical press. The Boulder Canyon Project, with its immense dam in Black Canyon, naturally interested him deeply, and he was a strong advocate of it.

In spite of failure of his physical strength, Mr. Entenman retained an undaunted spirit. One can imagine the bitter disappointment of a man, young in years, brought to the realization that all the force and brilliancy of his mind could not release him from the thraldom of ill health and that he must give up his cherished ambition to accomplish great things in his chosen profession; but never losing his courage, he faced the situation squarely, and accepted it as a strong man faces his fate, however hard it may be. The drawings for his Bee River Siphon were made while he was half reclining on his cot, yet the draftsmanship was such as would do credit to any engineering office. From the porch of his home he could see, over the tops of the almond trees, the summit of Mt. San Jacinto, generally snow-capped, and the tree branches were trimmed so that the outlook would be unobstructed. This glorious view of the majestic mountain gave him comfort and satisfaction.

It was an inspiring experience to spend an hour with him. His knowledge of subjects under discussion was so profound and his viewpoint so full of interest that one always came away feeling richer for the contact. He was fond of books and was a discriminating reader, analyzing and digesting what he read. He had a broad vision and his style of writing was clear, forceful and delightful. Many who knew him, like the writer, found it hard to understand why one gifted with such a wonderful mind, clear vision, natural leadership, and fine personality should be taken away, at such an early age, from the field of action where the qualities he possessed are so much needed. A friend speaking at his funeral services said, in part:

"Paul possessed a brilliant mind and had a great appetite for learning. He was a man of turbulent interests and subtle perceptions. He himself was brutally direct and liked others to be so. One had to brace one's feet to meet him. There was no chance to shirk or hide behind a graceful pose or a cultivated one, or any other kind of barrier between yourself and him.

"That was his genius—people became closer knit and more self-contained when he was around. He enjoyed life and had no listless pleasures. His mind was always active and this indefatigable creative energy sustained him through years of ill health, nor did it subside until the end.

"He felt the enthusiasm of discovery; he felt that existence was inspiring; he endeavored to extract the intrinsic from the accidental in love and beauty—in life and death.

"He stood his fate—racked in body, his soul was never sick.

"'I who have felt pain's tireless cohorts tread
Their wild reverberant march throughout my form,
Have learned to treasure torment and to dread
The childish instability, the warm
Futility of pleasure. None can know
Full ecstasy unless from pain and woe!'"

He was married on June 27, 1921, to Edith Burnette, of San Diego, Calif., who survives him. Through her wonderfully sympathetic understanding of him and his needs she was of great assistance to him in his illness, collecting information and making observations for him when they were required.

Mr. Entenman was elected a Junior of the American Society of Civil Engineers on March 1, 1910, and an Associate Member on February 4, 1913.

CHARLES CLARENCE FOSTER, Assoc. M. Am. Soc. C. E.*

Died December 27, 1928.

Charles Clarence Foster, the son of William E. C. and Laura Elisabeth (Miller) Foster, was born in Iowa City, Iowa, on December 23, 1880. He was graduated from the Iowa City High School in 1901 and from the State University of Iowa in 1904, with the degree of Bachelor of Science in Civil Engineering.

Soon after his graduation from the State University of Iowa, Mr. Foster entered the employ of the Chicago, Burlington and Quincy Railway Company, as Assistant Engineer, at Burlington, Iowa, where he accumulated valuable experience in maintenance-of-way and engineering practice. He left this position in 1907 to accept service as Resident Engineer with the Kansas City, Mexico and Orient Railway Company, on its Pacific Division, his headquarters being El Fuerto, Sinaloa, and, later, at Chihuahua, Chihuahua, Mexico. During this period he had charge of several miles of heavy railroad construction.

On his return to the United States in 1908, Mr. Foster was successively employed with the U. S. Drainage Investigation Survey; the W. J. Hoy Construction Company on building construction at St. Paul, Minn.; and the Great Northern Railway Company, in the capacity of Resident Engineer on construction. In 1911, he entered the service of the George J. Grant Construction Company, at St. Paul. He remained with this firm seven years, acting as Engineer and Estimator; he was also in charge of varied types of field and office work in connection with reinforced concrete buildings and structures.

During the World War, Mr. Foster acted as Assistant Engineer at the Army Supply Base, at Norfolk, Va., on the construction of Government buildings and warehouses. In 1920, he again returned to railroad work, entering the employ of the Pittsburgh and West Virginia Railway Company. He was in charge, as Resident Engineer, of the building of a branch line of that carrier into virgin coal fields of Pennsylvania and West Virginia, after which he devoted about two years to the engineering features in connection with opening a modern type of coal mine for the Cleveland and Western Coal Company, at Powhatan, Ohio.

In 1925, he moved to Pittsburgh, Pa., where he became associated with Messrs. John F. Casey and Company, General Contractors, in whose employ he continued until his death.

On December 25, 1909, Mr. Foster was married to Antoinette May Thompson, at St. Paul, who survives him.

He was a member of the First Presbyterian Church in Iowa City, and was also prominent in fraternal circles, as a member of the Phi Kappa Psi Fraternity, Iowa City Lodge No. 4, A. F. and A. M., Iowa City Chapter No. 2, Royal Arch Masons, Palestine Commandery No. 2, Knights Templars, and Osborne Shrine, St. Paul.

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* Memoir prepared by Hugh B. Holmes, M. Am. Soc. C. E.

Mr. Foster was of a modest and unassuming disposition—a most lovable and agreeable companion. He was exceedingly popular with all who came in contact with him, and to his friends and associates his death came as a surprise and severe shock. He died of double pneumonia on December 27, 1928, at his home in Bellvue, a suburb of Pittsburgh. Interment was made in the family plot at Iowa City, Iowa, on December 30, 1928.

For his co-workers, fellow members in the Society, and a host of others who knew and liked him, there is much sadness in the thought that so good a friend has departed. Mr. Foster was one who went about his work in a conscientious manner, giving much attention to his profession and but little to the gratitude, or the rewards, that might be due him. Those who came in close contact with him feel his loss keenly.

Mr. Foster was elected an Associate Member of the American Society of Civil Engineers on March 7, 1921.

EDWIN PHILIP FOX, Assoc. M. Am. Soc. C. E.*

Died April 18, 1929.

Edwin Philip Fox, the son of Mr. and Mrs. F. W. Fox, was born in Philadelphia, Pa., on March 19, 1881. Mr. Fox's educational training was received in the schools of Philadelphia.

His first position was that of Mechanical Draftsman with the Chicago Pneumatic Tool Company, which covered the period from March, 1900, to January, 1901. Mr. Fox then acted successively in the following capacities: From January, 1901, to February, 1902, as Structural Draftsman for the McClintic-Marshall Construction Company at Pottstown, Pa.; from February to September, 1902, as Structural Draftsman with Steel and Wike, at Philadelphia; from September, 1902, to July, 1904, as Structural Draftsman and Squad Leader for L. F. Shoemaker and Company, of Philadelphia; from July, 1904, to January, 1905, as Structural Draftsman for the American Bridge Company, at Pencoyd, Pa.; from January, 1905, to August, 1907, as Squad Leader for the McClintic-Marshall Construction Company, at Pottstown; from August, 1907, to March, 1908, as Squad Leader for the American Bridge Company, at Pencoyd; and from March to June, 1908, in a temporary engagement to look after drawing-room work on the Reventazen Bridge in Costa Rica for the Baltimore Bridge Company, of Baltimore, Md.

In June, 1908, Mr. Fox again entered the employ of the McClintic-Marshall Construction Company, in charge of structural detailing, estimating, and designing, standardizing of shop drawings, and, in addition, acting as Designing Engineer on general plant layout. He also had charge of detail design and of inspection of the structural work for the operating shields for tunnels under the East River, at New York, N. Y.

In all, Mr. Fox remained with the McClintic-Marshall Construction Company for a period of about twenty years, and it was on the recommendation of this Company that he went with the Wheeling Steel Corporation, at Wheeling, W. Va., to take charge of its structural work.

In April, 1924, Mr. Fox went to Cleveland, Ohio, with the Forrest City Structural Steel Company, as Chief Engineer and Vice-President of the Company. He remained with this Company until January, 1928, when he was made Assistant General Manager of the Petroleum Iron Works Company, at Beaumont, Tex., which office he held at the time of his death on April 18, 1929.

Mr. Fox was widely known as an authority on structural work. He was one of the founders of The Builders Exchange of Beaumont, Vice-President of the Gulf States Erection Company of Beaumont, as well as a member of the Chamber of Commerce and of the Rotary Club and the Beaumont Club. He was also a member of Centennial Lodge No. 544, A. F. and A. M., of Carnegie, Pa., and of the Lutheran Church.

Mr. Fox was married in 1903 to Annie B. Auchenbach, who, with one son and two daughters, survives him.

Mr. Fox was elected an Associate Member of the American Society of Civil Engineers on September 9, 1919.

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* Memoir prepared from information on file at the Headquarters of the Society.

GARRETT ALEXANDER FRASER, Assoc. M. Am. Soc. C. E.*

DIED APRIL 10, 1929.

Garrett Alexander Fraser was born on February 24, 1895, in Spokane, Wash.; his boyhood home, however, was in Butte, Mont., where he lived until he was seventeen years of age and had completed high school. His mother died in his late boyhood and a loving aunt, Margaret C. Gillette, gave him a mother's care and in return received his sincere and constant affection.

Mr. Fraser's schooling was practically completed at the outbreak of the World War in 1917, and the University of Washington at Seattle granted him a Bachelor of Science in Civil Engineering degree early in that year in order to permit him to prepare to enter the Army. His service record as prepared by the War Department the day following his death may well be quoted, as follows:

> "Garrett Alexander Fraser was admitted to the first Reserve Officers' Training Camp, Vancouver Barracks, Washington, May 15, 1917, organized under the provisions of Section 54, National Defense Act, where he trained, as a civilian candidate for a commission, until the close of the camp August 14, 1917, when honorably discharged.
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> "He enlisted on August 15, 1917, at Vancouver Barracks, Washington, as a private in the Engineers; was assigned serial number 2 439 404, and joined Headquarters, 316th Engineers, 91st Division, at Camp Lewis, Washington. He was appointed Battalion Sergeant-Major, September 13, 1917. He was in training at the Engineers Training Camp, Camp Lee, Virginia, December 28, 1917, to March 17, 1918, for a commission and was commissioned a Second Lieutenant, Engineers, March 18, 1918. He continued in training at Camp Lee, Virginia, until transferred in April, 1918, to Camp Dodge, Iowa, where he joined Company B, 528th Engineers, Service Battalion; sailed with it from the United States July 9, 1918, and served with this regiment in France, participating in the St. Mihiel offensive, September 12-16, 1918. He was promoted to First Lieutenant, Engineers, March 26, 1919. He returned to the United States May 30, 1919, and was honorably discharged at Hoboken, New Jersey, June 6, 1919, by reason of the demobilization of the emergency forces."

On his return home after demobilization Mr. Fraser attended the Butte School of Mines for a few months, later actually working in the Butte mines in the vicinity of the School. In January, 1921, and for the next eight years, he served on the staff of the Puget Sound Bridge and Dredging Company of Seattle, becoming a Vice-President in 1928. In these years, he was instrumental in bringing to a successful conclusion many engineering works of a general nature such as bridges, dredging, subaqueous tunnel, and a large multiple-arch dam, located in different parts of the United States, Canada. and Mexico. At the time of his death he had but recently moved to New York City and had become Assistant Secretary and Treasurer of the Metals Mining Shares, Incorporated, and of the Minerals Research Corporation. He is known best for having built the International Bridge across the Rio Grande between Brownsville, Tex., and Matamoras, Tamaulipas, Mexico, which was completed and opened for traffic for the Gateway Bridge Company on July 4, 1928.

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* Memoir prepared by Roy E. Miller, M. Am. Soc. C. E.

Mr. Fraser passed through his thirty-four years of life with all the loyalty, courage, and zest, and with all the joy and laughter that can come to healthy, clean-minded men of his day and generation, which is to say that none was more blessed. He was saved from bodily and mental injury in two years of service in France during the World War, only to be brought down to a death as sudden and dramatic as war itself: and yet it was a death that one who knew him intimately realized he would accept as wholly fitting to his life and thoughts—a death by an instrumentality of this new world in which he lived.

Mr. Fraser was killed when the airplane in which he was a passenger crashed in taking off at Tampico, Tamps., Mexico, on April 10, 1929. He was en route from the City of Mexico to Brownsville, Tex., and was one of five in the plane, all of whom were killed, so that none was left to tell the true cause of the crash. It is said, however, that planes were changed at Tampico, the change being hurriedly made and insufficient time being taken to properly warm up the engine, so that after making a height of perhaps 300 ft., the engine failed. Thereupon the pilot attempted to volplane back to the aviation field and in banking for the turn without power lost complete control of the plane. As in all other new lines of human endeavor, sacrifices seem necessary as a corollary to the advancement of the art of flying. It is difficult to take a philosophical point of view of this cost in human lives to accomplish progress in the development of the world's new machines when one loses a companion and a devoted friend.

Of the things that Mr. Fraser accomplished, none was so outstanding as the promise of future deeds. His life had been so largely one of preparation—his talents so versatile. His day had but just arrived.

His dominant traits were loyalty to his associates and friends; an abiding interest in all who came within his circle of acquaintanceship; and the ability to make himself understood and sincerely liked by every one. His mind was active and quick, and he had a full measure of initiative and energy. Because of natural modesty and because his main interests were in another direction, only a few of his companions knew that Mr. Fraser had rare mathematical ability which amounted to almost that of a genius. He belonged in that branch of the Engineering Profession from which business executives are raised.

Mr. Fraser was elected an Associate Member of the American Society of Civil Engineers on March 15, 1926.

EDWARD CHARLES GERSBACH, Assoc. M. Am. Soc. C. E.*

Died March 3, 1929.

Edward Charles Gersbach was born in Montezuma, Iowa, on July 4, 1880, the son of Frank and Emma (Schell) Gersbach. His father was born in Germany, but came to the United States at an early age. His mother was a native of Iowa, although of German extraction. He received his primary education in the town of his birth, and went from there to Iowa State College, Iowa City, Iowa, from which he was graduated in 1904 with the degree of Bachelor of Science in Civil Engineering.

After completing his High School course, Mr. Gersbach commenced work in engineering lines as Chainman and Rodman, first for the Iowa Central Railway Company, and, later, for the Chicago, Rock Island, and Pacific Railway Company. While he was in college, he spent his vacation in 1901 as Instrumentman for the Chicago Union Transfer Company, and the following summer as Inspector on sewer construction for the same Company. During the summer of 1903 he was Transitman for the Logansport, Hammond, and Chicago Traction Company, and following his graduation in 1904 was employed with the Marshalltown Interurban Railway Company as Transitman. He then served as Draftsman for the American Bridge Company, at Ambridge, Pa., for several months and, later, as Instrumentman with the Indiana Harbor Railroad Company.

In the meantime, Mr. Gersbach had taken a Government Civil Service examination. He passed this and accordingly was appointed to the U. S. Reclamation Service in May, 1905. During the next three years, he served in various capacities as Engineering Aid, Instrumentman, and Assistant Engineer. In January, 1907, he was in direct charge of the location and construction of 110 miles of lateral and waste ditches on the Huntley Project, in Idaho.

On being transferred to the Sun River Project, Montana, as Assistant Engineer, he had charge of the design and construction of concrete structures. In March, 1908, he was transferred to the Blackfoot Project, Montana, as Division Engineer.

In July, 1911, Mr. Gersbach entered the U. S. Indian Irrigation Service, where he was placed in charge of the construction of, and, later, the operation of, the Hogback Project on the Navajo Indian Reservation in Northwestern New Mexico. He retained this position until 1924, when he was transferred to work among the Rio Grande Indian Pueblos, with headquarters in Albuquerque, N. Mex., where he remained until his death.

His illness, which terminated fatally, was incurred in the line of duty. While he was engaged on instrumental work beside an open cut, he slipped, and, being a large man, injured one of the ligaments in his hip. Although he had the best of medical and surgical care, complications followed, resulting in his death on March 3, 1929. His body was taken to his old home in Montezuma for interment.

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* Memoir prepared by H. F. Robinson, Esq., Albuquerque, N. Mex.

During Mr. Gersbach's engagement on the Hogback Project, in New Mexico, he met Olive Johnston, who at that time was a missionary nurse at an Indian school, and they were married on April 3, 1912. They had one son, Frank. In addition to his widow and son, his surviving relatives are his father, a brother, Otto Gersbach, of Chicago, Ill., and a sister, Mrs. Nellie Barteld.

Mr. Gersbach was a careful and skillful engineer, respected and liked by his associates—a man with a host of friends. He was a Mason, with his affiliations in Montezuma. His death is a distinct loss to his community and the Department of the Government in the employ of which he had served so many years.

Mr. Gersbach was elected an Associate Member of the American Society of Civil Engineers on June 1, 1909.

WILLIAM CARY HATTAN, Assoc. M. Am. Soc. C. E.*

Died March 25, 1929.

William Cary Hattan, the son of Mark and Jennie E. (Siler) Hattan, was born at Kerrs Creek, Va., on December 15, 1875, of English (paternal) and German (maternal) ancestry. He was educated in the public schools and, later, attended Washington and Lee University, at Lexington, Va., from which he was graduated in 1899, receiving the degree of Bachelor of Science in Civil Engineering.

Mr. Hattan began his engineering work immediately after graduation, accepting his first position with the West Virginia Short Line Railroad Company as Rodman, Inspector of Materials, and Masonry Inspector. He was with this Company until March, 1901. From this date until August of the same year he was Masonry Inspector and Estimator for the Pennsylvania Railroad Company on the Wolf Creek Branch. From August, 1901, to January, 1902, he was in charge of a party doing general engineering work for J. H. Nichols, Civil and Mining Engineer, of Sharon, Pa. In January, 1902, Mr. Hattan accepted the position of Resident Engineer on construction with the Ozark and Cherokee Central Railway Company in Indian Territory. He was with this Company until January, 1903, at which time he returned to the East. From March to June, 1903, he was Assistant Resident Engineer on the Piney Extension for the Chesapeake and Ohio Railway Company, at Beckley, W. Va.

From June, 1903, until May, 1904, Mr. Hattan was associated with Vandevanter and Hood, Civil Engineers, of Baltimore, Md., in general engineering work, and as Assistant Engineer in charge of location and construction of the Baltimore and Belair Electric Railway. In May, 1904, he entered the employ of the Western Maryland Railroad Company as Resident Engineer on the construction of the Cumberland Extension improvement work. While in the service of this Company he had charge of the erection of several large bridges, concrete arches, station buildings, and water tank and pumping station.

In the fall of 1905, Mr. Hattan accepted a position as Resident Engineer with the Carolina, Clinchfield and Ohio Railway Company and was given charge of a residency at the summit of the Blue Ridge Mountain where the construction work was very heavy. Although this residency was only 4.4 miles long, it included six tunnels which required three years to complete. His services on this work were very satisfactory, and at its conclusion he was transferred to the south end of the Carolina, Clinchfield and Ohio Railway where he had charge of the erection of one of the largest bridges built by this Company. He remained with this railroad until the construction work was completed and the road put into operation in September, 1909.

In April, 1910, he returned to the Western Maryland Railroad Company as Resident Engineer on construction. His ability was noted by the contractors on this work and in July, 1910, the Carter Construction Company appointed him Division Engineer in charge of sub-contract work on the road. He held this position until December, 1911.

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* Memoir prepared by C. K. Lucas, Esq., Erwin, Tenn.

When the Carolina, Clinchfield and Ohio Railway Company decided to build its Elkhorn Extension Mr. Hattan was made Division Engineer and placed in charge of the construction work on this 32-mile branch which, on account of the topography of the country and its inaccessibility, presented some very difficult engineering problems. On this extension there were eight steel and concrete bridges and twenty tunnels with an aggregate length of 20 786 ft., one of which, Sandy Ridge, was 7 854 ft. long, and lined throughout with concrete. This extension cost approximately $5 500 000, and upon its completion in June, 1915, Mr. Hattan accepted a position with the Kingsport Improvement Corporation. While with this Company he had charge of the municipal development and laid out the larger part of the present city of Kingsport, Tenn.

In June, 1916, Mr. Hattan entered the service of Thomas and Jones, Contractors, as General Superintendent in charge of construction work on the Missouri, Kansas and Texas Railroad Belt Line and its terminals at San Antonio, Tex. Upon the completion of this work in September, 1917, he took charge of the construction of Park Aviation Field, near Memphis, Tenn., where he was General Superintendent for the contractors, Thomas-Harmon Company. From February until November, 1918, he was Area Superintendent in charge of Area "C", U. S. Government Explosives Plant, at Nitro, W. Va., Thompson-Starrett Company, Contractors.

In December, 1918, he moved to Meyersdale, Pa., where he was engaged in private practice as Engineer and Contractor until July, 1920, when he accepted the position of Chief Engineer for the Carolina, Clinchfield and Ohio Railway Company (now the Clinchfield Railroad Company), with headquarters at Erwin, Tenn. This Company was very fortunate in securing the services of Mr. Hattan, as he proved to be a most efficient and accurate engineer with varied experience in both construction and maintenance work. He remained with this Company as Chief Engineer until his death which was caused by complications following an operation for appendicitis.

Mr. Hattan was a modest, unassuming, Christian gentleman who commanded the admiration, respect, and friendship of all who came in contact with him. He was an Elder of the Presbyterian Church; Assistant Teacher of the Brotherhood Bible Class; President of the Kiwanis Club of Erwin; and a member of the City School Board. He was also a member of the Tennessee Society, Sons of American Revolution, and of the American Railway Engineering Association.

He was married at Meyersdale, Pa., on April 12, 1912, to Sara Stein who, with a son and daughter, survives him.

Mr. Hattan was elected an Associate Member of the American Society of Civil Engineers on November 6, 1907.

JOHN LUCEY, Assoc. M. Am. Soc. C. E.*

Died September 16, 1929.

John Lucey was born in Little Falls, N. Y., on June 24, 1886, the son of the late Cornelius D. and Bridget (Lynch) Lucey.

Mr. Lucey received his early education at the public schools of Little Falls and was graduated from St. Mary's Academy, Little Falls, N. Y., in 1898. He attended Union College at Schenectady, N. Y., for a time and, later, the Engineering School of New York University from which he was graduated in 1908 with the degree of Bachelor of Science in Civil Engineering.

He began his engineering career in the Construction Department of the New York Central Railroad Company as Rodman and Transitman, making surveys and plans for the track changes in connection with the Barge Canal crossings, in September, 1908. In 1914, he was made Assistant Engineer in charge of construction of concrete arches and Office Assistant in charge of plans and final estimates of this work.

Mr. Lucey was employed by the Rockwood Sprinkler Company of New York City from May, 1914, to September, 1916, in making estimates and designing automatic sprinkler equipment.

He then returned to the New York Central Railroad Company as Assistant Engineer and, in 1916, and 1917, was in charge of the construction of a 250-ft. span, double-track, steel, truss bridge over the Barge Canal at Brewerton, N. Y., including track changes and water supply facilities.

In May, 1917, Mr. Lucey left the United States for the Dominican Republic, West Indies, to take charge as Resident Engineer, under the direction of the Department of Public Works, of harbor improvements in the Port of Puerto Plata. This work included the dredging of the harbor and the construction of a reinforced concrete pier on concrete piling with a custom house of similar material and a steel warehouse built on the pier.

For the next three years he was District Engineer, in general charge of all Departmental work in the District of the Cibao. This work consisted principally of the construction of about 200 km. of macadam highway; the surfacing and graveling of 90 km. of old dirt road; and the location, grading, and surfacing of 110 km. of new road. The greater part of this work was done by force account, Mr. Lucey being directly responsible for the organization, the direction of the work, and the handling of the funds for the payment of all labor and materials. All this was done to the entire satisfaction of the Dominican Government officials.

From 1921 to 1923 he was in the City of Mexico, Mexico, as Superintendent on the completion of a three-story concrete and steel building for the Compania Importadora del Auto Universal and on a hydro-electric survey for the Real del Monte Mining Company. From 1924 to 1926, he was with the Mexican Railroad Company as Chief of Party, under Mr. O. G. Bunsen, on the location of the proposed line between Pachuca and Tampico.

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* Memoir prepared by John T. Collins, Assoc. M. Am. Soc. C. E.

Mr. Lucey then went with the International Railways Company of Central America in Salvador, and for two years, until the end of 1928, was in charge of the location and construction of the railroad for the Polichic Banana Company and as Supervising Engineer of the line between Zacapa and Ostua for the section being built under contract by Keilhaue and Rodezno.

Outside his professional life John Lucey turned to literature for his real recreation. From his childhood until his death he explored the realm of books. The telling of what he had discovered, leavened with his keen sense of humor, made of him a very enjoyable and entertaining associate. Besides his companionship, he was a true friend, always doing thoughtful things for those whom he liked. His death was a great loss to his many friends, his engineering associates, and to his family.

He never married. He is survived by five sisters: Mrs. Mary Stewart, of Hollywood, Calif., and the Misses Katherine, Anna, Julia, and Theresa Lucey, of Yonkers, N. Y.; and one brother, Jeremiah Lucey, of Pittsfield, Mass.

Mr. Lucey was elected an Associate Member of the American Society of Civil Engineers on January 13, 1919.

CLIFFORD LYNDE, Assoc. M. Am. Soc. C. E.*

Died May 21, 1929.

Clifford Lynde, the son of James and Lizzie E. (Clifford) Lynde, was born at Chelsea, Mass., on November 16, 1884.

After graduating from the Massachusetts Institute of Technology in 1906, Mr. Lynde was located in Oil City, Pa., for a year. From there he went to the New York Board of Water Supply as Assistant Engineer, with headquarters in Walden, N. Y. He was employed on the construction of the Catskill Aqueduct until 1914, when he moved to Brooklyn, N. Y., where he was engaged on subway construction by the Cranford Company. At the completion of this work, he was employed for a short time by Prentice Sanger, Architect, returning to Walden in 1918 to make special studies for proposed new equipment for the Walden Knife Company.

During this period, Mr. Lynde suffered an attack of influenza and, later, rheumatic fever with resultant heart trouble, from which he never completely recovered. For many months he made a brave struggle to regain his health and, in an effort to obtain strength, he moved to Miami, Fla., where he resided from 1923 to 1928. He established himself in business as a Manufacturer's Agent and was very successful in this venture, surviving the boom and deflation times as well as the great hurricane. Because of failing health, he again returned north to spend six months in a Montclair, N. J., Sanitarium. He survived until May 21, 1929, when he passed away in Walden, N. Y.

Mr. Lynde was married to Mildred Fowler, in Walden, on June 11, 1912. He is survived by his widow, his mother, and two children, Elizabeth A. and Fairfield F. Lynde.

"Cliff" will long be remembered by his many friends for his genial and happy disposition, his unfailing optimism, and his ability to surmount cheerfully for years the greatest of obstacles—ill health. In his passing, those who counted themselves among his friends have sustained a real and genuine loss.

Mr. Lynde was elected a Junior of the American Society of Civil Engineers on December 1, 1908, and an Associate Member on November 28, 1916.

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* Memoir prepared by Dean G. Edwards, M. Am. Soc. C. E.

CHARLES DIX McARTHUR, Assoc. M. Am. Soc. C. E.*

Died November 28, 1929.

Charles Dix McArthur was born in Plattsburgh, a small town near Springfield, Ohio, on August 23, 1870. His father, Smith McArthur, of Scotch ancestry, was a grandson of Duncan McArthur, Governor of Ohio. His mother, Phoebe (Judy) McArthur, was of Swiss lineage. His early education was in the public schools in Plattsburgh and New Carlisle, Ohio, and, later, he had an opportunity to attend Antioch College for two years, but the necessity of earning his own living prevented his graduation.

In 1896, while working for the Eastman Kodak Company of Rochester, N. Y., Mr. McArthur took advantage of the night course in machine design at the Mechanics Institute in that city from which he received a diploma in June, 1900. From 1897 until 1901 he was also Instructor in Mechanical Drawing at the Institute. In August, 1899, and during the year following, he was engaged in designing metal working machinery for the Gleason Tool Company. He resigned this position, however, to enter the employ of the Buffalo, Rochester, and Pittsburgh Railway Company, for which he designed the mechanical equipment of the locomotive repair shops at Dubois, Pa., and also superintended the erection of the equipment of these shops. In 1903, Mr. McArthur went as Master Mechanic to the Adrian Blast Furnace, at Dubois, removing to New York City in September, 1903, to become City Manager of the heating and plumbing business of Howe and Bassett. He spent several years in the heating business, during which he both designed and installed heating equipment in various buildings, notably the Stuyvesant High School.

In January, 1908, Mr. McArthur designed and applied for a patent on a steel form for use in the construction of the Catskill Aqueduct. This led to his employment as Chief Engineer by the Blaw Collapsible Steel Centering Company of Pittsburgh, Pa., which, later, became the Blaw-Knox Company. He remained with this company for many years, except for a brief interlude during 1913 and 1914 when he was employed by Stewart and Fauguier in the construction, in from 30 to 70 ft. of water, of the quay walls of the first unit of the Halifax Ocean Terminals for the Intercolonial Railway Company of Canada; he designed the forms for this construction work. While he was with the Blaw-Knox Company, Mr. McArthur took out more than twenty patents relating to steel forms for the construction of aqueducts, railway, and other tunnels, subways, retaining walls, and similar structures. He was a familiar and highly regarded figure wherever important concrete works were being designed or constructed. During most of these years Mr. McArthur lived in Pittsburgh, but he represented the Company for a time in London, England, and also in California.

In 1928, he resigned from the Blaw-Knox Company for the purpose of devoting his entire time to the development and introduction of new woodworking machines which he had invented, and on January 2, 1929, the Mc-

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* Memoir prepared by William Mayo Venable, M. Am. Soc. C. E.

Arthur Company was incorporated with its office in Dayton, Ohio, for the purpose of manufacturing "undercut" boring machines, "metal dowls", and other patented specialties. This Company had just been brought to a profitable status when on Thanksgiving Day, November 28, 1929, while on a pleasure drive to dine in the country, he died quite suddenly and unexpectedly.

Mr. McArthur was always greatly interested in new engineering ventures and kept himself informed as to the latest developments in the Engineering Profession. He possessed a strong and forceful character, and his work in his chosen profession was well done. He was loved and respected for his faithfulness, loyalty, and honesty by all with whom he came in contact. He loved music, books, flowers, and little children, the simple life, and the beauty of Nature. He liked to shape things with his own hands, was a finished workman in wood and metal, and adorned his home with cabinet work and carvings of his own making.

He is survived by his widow (*née* Therese M. Ridenour) and a son, Arthur, as well as a son, Duncan, by his first wife (*née* Louise Middledorf), of California. He is also survived by five sisters and two brothers.

He was a member of the Protestant Episcopal Church of the Redeemer in Pittsburgh, Davage Lodge (Masonic), Pittsburgh, the American Association for the Advancement of Science, the American Association of Engineers, and the Engineers' Club of Dayton.

Mr. McArthur was elected an Associate Member of the American Society of Civil Engineers on January 17, 1916.

FRANK HILL McGUFFIE, Assoc. M. Am. Soc. C. E.*

Died October 17, 1928.

Frank Hill McGuffie, the son of Matthew and Martha (Morgan) McGuffie, was born at Waverly, N. Y., on August 27, 1887. His early life was spent in his native village. He attended St. John's College, Manlius, N. Y., from 1903 to 1906, and for several months in 1913 was a student at Cornell University, Ithaca, N. Y., in special engineering work.

During the summer months, while he was at college, Mr. McGuffie worked as Chainman and Rodman for the Lehigh Valley Railroad Company, and continued in that employ as Rodman and Levelman from 1906 until 1908. In July, 1908, he accepted a position as Engineer in charge of construction on the estate of Mr. B. L. Winters, at Smithboro, N. Y. His work there which continued until October, 1910, consisted of the design and construction of roads, small bridges, and a water supply system. He then spent several months as Draftsman and Transitman on base line surveys for the Riordan Company, of London, Ont., at St. Jovite, Que., Canada.

In May, 1911, Mr. McGuffie entered the service of the New York State Department of Highways and for more than six years was engaged as a Levelman in charge of contracts and survey parties on location. From 1918 to July, 1924, inclusive, he served as Assistant Engineer with the same Department in responsible charge of the design of pavements and bridges. His duties included supervision of village and county paving, highway construction work, and the maintenance of 150 miles of highways. Coincident with this engagement, Mr. McGuffie was County Engineer of Sullivan County, New York, for three years. He also did a certain amount of private work, including the design of a sewerage system for the Village of Waverly, bridge work for the Delaware and Northern Railroad Company, and the design of a dam for the Callicoon Electric Company.

From August, 1924, until his death, he was employed by the National Surety Company of New York as Field Engineer, with headquarters at Denver, Colo. He was sent West by the Surety Company to supervise the completion of a large paving job for the Colorado State Highway Department on which the contractor had defaulted. His work for this company included the making of cost estimates, preparation of plans and specifications, acting in an advisory capacity to contractors, and the compiling of contract data for underwriters. His territory included Colorado and the adjacent States, where he was well and favorably known by both contractors and engineers.

Mr. McGuffie died at Denver on October 17, 1928, and was buried at Sayre, Pa., which is near his old home. Although troubled with pernicious anæmia to some extent, he was apparently in good health and was active in his duties up to a day or two before his death, which came as a distinct shock to his family and friends.

Because of his genial disposition and pleasing personality he was popular among his friends and business associates. He was generous and, with a natural inclination to be accommodating, was willing at all times to take endless trouble in order to be of service to a friend. Gifted with much ability and energy he gave promise of attaining a considerable degree of success in his profession, had not death terminated his career at a relatively early age.

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* Memoir prepared by William B. Freeman, M. Am. Soc. C. E.

Immediately on his election to the Society, less than a year before his death, Mr. McGuffie became actively interested in the affairs of the Colorado Section and had demonstrated that he was a valuable addition to the membership. He was also a regular attendant at the meetings of the Cornell Club of Colorado. He was a member of the Masonic Fraternity, a Thirty-second Degree Mason, and a Shriner.

On May 24, 1928, he was married to Reba Burrus, at Denver, Colo. In addition to his widow, he is survived by a sister, Mrs. Lois C. Davenport, of Sayre, and a brother, Dr. Robert N. McGuffie, of Passaic, N. J.

Mr. McGuffie was elected an Associate Member of the American Society of Civil Engineers on December 5, 1927.

THOMAS McKENZIE, Assoc. M. Am. Soc. C. E.*

Died July 25, 1929.

Thomas McKenzie, the son of Andrew and Martha (Cook) McKenzie, was born in Providence, R. I., on September 10, 1867. His parents were of Scottish and English ancestry, his father's family having come to the United States in their youth. His mother died in his childhood and he went to live with his uncle and aunt, Mr. and Mrs. John McCulloch, of Providence, R. I.

While Mr. McKenzie was still a young man his uncle and aunt removed to Westerly, R. I., where he attended the local public schools and, later, learned the moulder's trade in the shops of the C. B. Cottrell and Sons Press Works, in Pawcatuck, Conn., situated across the river from Westerly. His uncle, who also was employed there as a Master Moulder, later, decided to enter the business field in Westerly for himself, taking his young nephew with him. Mr. McKenzie continued in business with his uncle until 1889, when he determined to abandon foundry work and enter the engineering and building profession by means of which he might work his way upward in life to a more satisfactory and lucrative career.

In January, 1889, he went to Providence, and entered the office of the late J. Herbert Shedd, M. Am. Soc. C. E., as a student in civil engineering. He remained with Mr. Shedd for three years, acting as Rodman, Chainman, and Office Draftsman. During his studentship he attended the night schools in Providence, employing his spare time improving his natural talents as a Draftsman, in which work he later became very proficient.

While Mr. McKenzie was with Mr. Shedd, from 1889 to 1892, he was engaged on the encroachment line surveys on the Pawcatuck River at Westerly, and Stonington, Conn., as well as on other engineering work in connection with Mr. Shedd's general practice.

After completing his three years' apprenticeship, he found it necessary to make a change of employment, owing to Mr. Shedd's connection with the City Engineer's Office in Providence. Consequently, early in 1892, he entered the office of the late Samuel M. Gray, M. Am. Soc. C. E., as a Construction Engineer. Mr. McKenzie remained with Mr. Gray for five years as an Assistant Engineer. During this time he was in charge of construction at Bradford, Vt., Chautauqua, N. Y., Meredith, N. H., Gallipolis, Ohio, and at Rockville, Conn.

His work at Bradford consisted of a small water-works system, including a storage and distributing reservoir. At Chautauqua, he completed the sewer system already well under way at the time of his arrival on the ground. This work included the building of a Chemical Precipitation Disposal Plant, which kept him busily engaged all of one winter. This was followed by the building of a water-works system, including a small storage and distributing reservoir, with its earth work dam and masonry spillway, at Meredith. At Gallipolis, he constructed the water-works plant, including a system of pipes and hydrants, reservoir, pumping station, and a unique source of supply. This consisted of sinking four 4-ft., cast-iron pipe wells on an island in the Ohio River. These wells were connected with the pumps by a long line of suction pipe located on the high banks of the river. Considerable difficulty was experienced in the construction of these works, but they were finally completed in a satisfactory manner. His work at Rockville was the building of a sewer system in 1895 and 1896. This construction required considerable rock excavation in the streets and an inverted siphon under the Hockanum River.

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* Memoir prepared by George H. Leland, M. Am. Soc. C. E.

In the construction of these various undertakings, Mr. McKenzie was very careful and conscientious to see that all parts of the work were done in accordance with the contracts and specifications.

In 1897, the Town of Westerly, after considerable litigation in the Courts, acquired the rights, franchises, good-will, and title to all the property held by the Westerly Water Company, including the right to use the waters of Shunoc Brook in Connecticut as a source of water supply and the privilege of furnishing water to the Village of Pawcatuck, Conn.

Under the direction of Mr. Gray, Consulting Engineer, plans were made to install a system of small wells driven in the Pawcatuck Valley. These plans involved the abandonment of Shunoc Brook as a water supply, and necessitated a new pumping station and new mains. Mr. McKenzie was selected to take charge of the reconstruction of the new and improved system. After completing the improvements and additions, the system was given a thorough test, found to be satisfactory, and the works were then turned over to the Town. The Board of Water Commissioners immediately appointed Mr. McKenzie to be Superintendent and General Manager for the Westerly Water-Works which position, extending over a period of more than thirty-two years, he held at the time of his death.

During his early connection with the water-works, or in 1902, the pipe system was extended to the Watch Hill District and along the South Shore, supplying the summer population with good water and fire protection. The extension of the distribution with the ever-increasing consumption of water on both sides of the river required more storage capacity than was available from the one small iron stand-pipe, built in 1888 by the Water Company. As a result, one of the most important improvements in the system was the building of a reinforced concrete stand-pipe under the supervision of Mr. McKenzie, with Mr. Gray as Consulting Engineer. The plan was successfully completed by the Aberthaw Construction Company, of Boston, Mass., and was put into service in connection with the iron stand-pipe then in use.

In 1914 the Town decided to install a sewer system for the Westerly Fire District and Mr. McKenzie was instructed to take charge of the surveys for this work in connection with his other duties as Superintendent of the Water-Works. The sewer system as finally planned for the District included in addition to the pipe system, two pumping stations, an Imhoff tank, and two filter beds, costing approximately $600 000. These works were built largely by contract under the supervision of Mr. McKenzie as Engineer in Charge.

In connection with his duties attending the water-works and sewer system of Westerly, he found time to rebuild some of the streets and highways for the town, also, to assist in an advisory capacity some of the near-by towns and villages with their water problems. He was an indefatigable worker, and was always willing to serve in the interests of the town's affairs, and, later, in the building of the new Memorial Hospital in Westerly, for which he served on the Building Committee.

Mr. McKenzie was prominent in Masonic circles, having reached the Thirty-second Degree several years previous to his death. He was Grand Master of the Connecticut Grand Lodge. In 1916, he was Connecticut State Chairman of the George Washington National Memorial Association, besides holding many other important offices in Rhode Island and Connecticut. He was a member of the New England Water Works Association and the Providence Engineering Society. He was a communicant of Christ Protestant Episcopal Church, of Westerly, a member of the Vestry, Junior Warden, and Treasurer of the Church organization.

In 1892, while working on his first construction job with Mr. Gray at Bradford, Vt., he met Gertrude F. Jones, daughter of the late Dr. Julian F. Jones. They were married in Bradford, Vt., in January, 1896, and made their home in Providence until Mr. McKenzie went to Westerly in 1897. His wife died at her home in Westerly on September 28, 1928, leaving one son, Julian L., 22 years of age.

Mr. McKenzie felt the loss of his companion very keenly, but bore up bravely and plunged into his work far beyond his strength, which over-activity brought on a severe attack of heart trouble terminating in his death after four days' illness. He always had been of robust health and unusually well and strong, but he over-taxed himself.

He was very systematic in all his work; careful statistics and records were kept of the operation of both the water and sewer plants, carefully indexed, and put on file, together with the plans of all the works. He always managed to surround himself with a corps of faithful assistants and co-laborers, who at all times were willing and ready to perform the tasks he gave them. Many of these assistants and laborers remained with him for several years. Mr. McKenzie was greatly respected and admired as a faithful and loyal servant of the community in which he served, was popular as a speaker at public meetings, and an able executive and presiding officer. His loss will be greatly felt in the town in which he spent the best part of his life.

In closing, it may be well to remark that Mr. McKenzie owed a large measure of his success and formation of character to his Aunt Mary who cared for him in his early life in place of his mother and of whom he was very fond. His own mother could not have done more for him. Later, he repaid his debt of gratitude and love to her by taking her into his own home in her declining years.

Mr. McKenzie was elected an Associate Member of the American Society of Civil Engineers on November 6, 1895.

FREDERICK MEISTER, Assoc. M. Am. Soc. C. E.*

Died April 1, 1928.

Frederick Meister was born in Jersey City, N. J., on July 5, 1881, the son of Frederick and Anna Meister (*née* Memple). His father was one of the earliest settlers in the Heights Section of Jersey City and one of its pioneer builders. In his son's boyhood recollections, building operations and developments incident thereto (in the course of which an almost suburban community became an important section of a growing city) played a major part, so that one might say that in his youth he was dedicated to the profession which was to have such great import in his life.

His father often spoke of the intense interest the lad displayed in everything that pertained to building and decided that should this inclination persist, he would have him educated as an Architect. A marked talent in drawing further emphasized the advisability of a pursuit along these lines, and, at the age of nine, the boy began the study of drawing and artcraft under private masters. Previously, he had been enrolled as a pupil in the then flourishing private school in Jersey City Heights known as "The Academy", where he acquired his scholarly perfection of the German language, an invaluable asset in later years when the study of engineering became paramount, for it gave him the advantage of access to the best that the German schools of technology afforded. He was an earnest student of the piano and after several commendable attempts at composition, almost decided upon music as his career. After graduating from the public and High Schools of his native city, however, he was admitted to Pratt Institute, at Brooklyn, N. Y., two years in advance of the prescribed age on the merit of his drawing skill and advanced knowledge of mathematics.

In 1898, Mr. Meister received a degree in Architecture from Pratt Institute and thereupon became apprenticed to Emil Guhl, pre-eminent as an Architect at that time in Jersey City. Here was a master of the old school, whose disciplining was more that of the schoolmaster than of the employer. Mr. Meister regarded this particular period of association as one of the happiest of his life, with so much of mischief and recreation included in the day's routine.

His work consisted in surveying and laying out many of the North Hudson towns, and this outdoor occupation was more pleasant than lucrative, however little one relished dragging about the necessary equipment. Back at the drawing board in Mr. Guhl's office, the prospect of practicing architecture in a small community, with the work shorn of its beauty and much of the vision so necessarily a part of this fine profession, became irksome indeed. Consequently, with the introduction of steel which was destined to play such a large part in the building industry in New York, Mr. Meister turned to the study of structural engineering and found his first engagement in this field with Messrs. Levering and Garrigues, by which firm he was employed for seven years. He severed his connection with this Company to take up the duties of Chief Draftsman, and, subsequently, Chief Engineer, with the Hinkle Iron Company, of New York, N. Y., with which he was associated for more than twenty-two years, or until the Company retired from active business. From the former employees of this latter concern sprang the present Hinkle Steel Construction Company, of which organization Mr. Meister was Vice-President and Engineer at the time of his death.

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* Memoir prepared by Edward J. Spoerer, Assoc. M. Am. Soc. C. E.

For many years preceding his death his services as Consulting Engineer were sought by the leading architects and builders of his vicinity, among them being Thomas Lamb, Townsend, Steinle and Haskell, Eugene DeRosa, Incorporated, Jacobs and Youngs, Shapiro and Sons, H. B. Moss, Richard Carvel, the Cova Realty Company, Kenlon and Michels, C. H. Horn Sons, George Kiester, and Cohn Brothers, of New York. He also worked with William Neumann, J. Welitoff, and Flagg and Stades, of Jersey City (all architects), serving as Engineer of Record on all their projects.

In the construction of theatres and auditoriums in which he specialized and in which he earned a reputation second to none, his services were sought throughout the United States, and many aspiring engineers were glad of the opportunity to work with him on any project, for the knowledge they gained by such an association. His adeptness in structural problems was almost uncanny; nothing baffled him and no problem failed to meet defeat at his hands. His opinion in matters of arbitration were accepted as conclusive and beyond question. In the building departments he was affectionately known as "The Meister". The name was not inaptly applied, for he gave of his knowledge and services without stint, a real example of a "Master" and a Christian gentleman. Great in body, great in soul, and great in mind, he did much to help others find themselves, and felt well repaid in the pleasure of being of service. It was this bigness of nature, this uncomplaining acceptance of the burdens of others, to which may be attributed the heart attack which caused his sudden death. So rugged was his constitution that he never feared for himself, and practically his last act was a needless sacrifice of himself for the benefit of his associates.

Mr. Meister's interest in theatre construction was so engrossing that he turned his efforts to the abolishment of the disturbing posts which supported the balconies of all theatres and large auditoriums. On November 25, 1913, he received Letters Patent on "Floor Supporting Structure of Theatres and the Like" which eliminated for all time the necessity of unsightly posts to support balconies. He allowed this fine innovation to be used by architects throughout the United States for no other consideration than the asking. In fact, he had gained such prominence as an authority on theatre construction that at the time of the tragic collapse of the roof of one of the largest theatres in Washington, D. C., he was the one first selected by the authorities to conduct an investigation into the cause of the catastrophe. Unfortunately, the serious illness of his father, to whom he was deeply devoted, would not permit of his protracted absence and he was obliged to refuse the commission. Thereafter, many municipal authorities throughout the country sought his opinion and guidance in an inspection of their theatres and auditoriums.

Quiet, unassuming, modest to a fault, Mr. Meister often allowed the credit of a worthwhile feat to accrue to some one to whom it might prove a stepping stone to higher things. It is rarely that at a man's bier men of high and lowly station meet on common ground to weep openly for a loved associate and friend. These stepping stones should ofttimes bring to mind a most lovable character and distinguished gentleman, whose untimely demise is a real loss to the Engineering Profession, for it is problematical to what exceptional attainments his genius might have led had life been spared to him.

Among Mr. Meister's engineering works might be mentioned the following: Hammerstein's former Lexington Avenue Opera House and his personal services to Oscar Hammerstein during the erection of the Manhattan Opera House; the O'Shea Theatre, Toronto, Ont., Canada; the Harrisburg and the State Theatres, Harrisburg, Pa.; the Strand and Capitol Theatres, New Britain, Conn.; the Danbury Theatre, Danbury, Conn.; the Waterbury Theatre, Waterbury, Conn.; the Strand and York Theatres, York, Pa.; the Ritz Theatre, National Theatre, and Capitol Theatre, Jersey City, N. J.; several theatres for B. S. Moss, New York City; the Colonial Theatre, Bethlehem, Pa.; the Audubon Theatre, New York City; the Hyde Park Theatre, Hyde Park, Pa.; New York Public School No. 178; the Chamber of Commerce, Spingarn Building, Lerner Building, and Richards Chemical Building, Jersey City, N. J.; the Elks' Clubhouse, Asbury Park, N. J.; the Cova Office Building, 60th Street and Broadway, New York City; a Roman Catholic school, Mariners' Harbor, Staten Island; Park Avenue Hotel, 38th Street and Park Avenue, New York City; the Borden Plant and the Griffith Piano Building, Newark, N. J.; and the New Bank Building, Hackensack, N. J.

In addition, work on which he was engaged as both architect and engineer also may be mentioned: the Rockaway Hotel and Park Inn Baths, Rockaway, N. Y.; the Service Station of the Glidden Motor Company, New York City; the Distributing Station, Lotus Oil Corporation, New York City; the garage for the Automobile Club of America; the service station for the Stewart Distributing Company, New York City; and numerous apartment houses and office buildings in New York City and Jersey City. His last and perhaps most representative work is the present additions to the Cathedral of St. John the Divine, New York City, a task to which he devoted four years, surmounting many problems which will attest through the ages to his splendid ability.

The following are excerpts from many beautiful expressions of sorrow on Mr. Meister's passing:

Joseph F. Dujat, Sculptor:

"Had I command of all the words in every language they would still be inadequate to express my sorrow. It is impossible to express in words the extraordinary high esteem in which I held the friendship of Frederick Meister, whose superb character made one better for having known him, and his spirit will continue to permeate the actions of those with whom he came in contact".

The Rev. Father A. J. Molinelle:

"It was a blow to me to learn of his sudden passing, so much so that it did not seem true. When I saw him last he appeared to be strong and healthy and full of vigor and ambition. I had occasion to meet him off and on and consulted him on construction in general, and have always found him faithful and true to me and right and proper in everything which he undertook. I feel his loss more than I can tell."

James E. Geissberger, Civil Engineer:

"He will live in my memory always. I have lost a loyal, a noble friend, the community a citizen of sterling character, and the profession a brilliant mind, so he has not lived in vain! His motto, which he religiously lived up to was, 'Helpful, honorable, and true be man's purpose.' ".

M. Shapiro and Sons, Engineers and Contractors, New York City:

"We have all worked with Mr. Meister these many years on many of the theatres in New York, and during this period we have found our association with him one of the greatest pleasures of our work. To him we owe a great deal. He was generous in his advice because it seemed to give him joy to be of service. He will be mourned by nearly every Engineer in New York who is connected with theatre construction."

The late Canon Robert Ellis Jones, of the Cathedral of St. John the Divine, New York City:

"I admired him greatly for his professional attainments and his fine character. His deep interest in our Cathedral was a special assistance to me personally. When our building is finally complete it will owe much of its individuality and majesty to Mr. Meister's guidance—a relation that I will not allow to be forgotten".

In conclusion, his widow, the former Ottillie C. Kerner, of Jersey City, N. J., to whom he was married in 1907, and who survives him, feelingly eulogizes him in the following lines:

"REQUIEM: FREDERICK MEISTER

"What I am now is Mortal's Fate:
Life's purpose ended: Naught to make
My presence felt in your dear midst—
Unless it be, a thought of me
Vibrant and eager to give my best
To help and serve: That is the test
Of Man's short sojourn: Have I won
That recognition, now that I am done?
Full well content with Life am I
This priceless bond which shall not die!
Your kindly thought shall be to me
The shroud of Love in which I sleep;
Oblivion shall not penetrate
This gossamer veil whose watch you keep:
Until that Hour, when you, dear Friend,
Shall with me in God's realm attend!
Until that day, fast be the tie
By which no Man can really die;
For Death is conquered so combined:
In Memory's Stronghold fast enshrined."

With many engineering feats as living memorials to his genius, his finest monument will ever be in the hearts of those who called him "friend".

Mr. Meister was elected an Associate Member of the American Society of Civil Engineers on June 23, 1916.

EMIT COIN NICKEY, Assoc. M. Am. Soc. C. E.*

Died May 5, 1929.

Emit Coin Nickey, the son of L. F. and Addie M. Nickey, was born near Trafalgar, Ind., on March 14, 1882. He was educated at Ohio Northern University where he completed a partial course in 1904.

After leaving the University Mr. Nickey's first work was with the St. Louis-San Francisco Railroad Company as Rodman and Instrumentman until June, 1905. He then accepted a position with the County of Butler, Missouri, as County Surveyor, and was also engaged on townsite subdivision work in Southeast Missouri.

From June, 1905, until May, 1928, Mr. Nickey served as County Highway Engineer of Butler County, Missouri, having charge of all highway and bridge improvement work of that county and the financing of the same. In August, 1919, he successfully waged a campaign for a $500 000 county bond issue for roads and bridges, to meet Federal and State Aid funds for the 100-mile road system in Butler County completed at a total cost of approximately $1 000 000. In addition, he financed general county funds for road and bridge improvements each year approximated at $100 000 and prepared plans, estimate of cost, and made location surveys for all this work.

From August, 1913, until September, 1915, he served as Drainage Engineer in charge of surveys. He prepared plans and estimates of cost for Drainage Districts Nos. 10 and 11 in Butler County, and also for more than 40 miles of ditches. The cost of this undertaking was approximately $130 000.

From May, 1905 to June, 1909, Mr. Nickey served as City Engineer at Poplar Bluff, Mo. Again, from January, 1927, until September, 1928, when he resigned, due to ill health, he was City Engineer of Poplar Bluff, in charge of all the engineering work within the city, including a street paving program estimated at $400 000. He also was engaged on sewerage and other drainage work. From 1905 until his death in 1929 he served as County Surveyor of Butler County.

Mr. Nickey was a man of high ideals, of sterling character and of the highest ability. Of pleasing personality, he possessed a genial disposition, endearing himself to all who knew him. He was a member of the American Association of Engineers, and a member of Lodge No. 209, A. F. and A. M., of Poplar Bluff. He was also a member of the Benevolent and Protective Order of Elks, Poplar Bluff Lodge No. 589.

Mr. Nickey moved from Missouri in October, 1928, to Colorado where he died on May 5, 1929, of cerebral hemorrhage. The funeral rites were held under the auspices of the Colorado Springs Lodge No. 76, A. F. and A. M., and Colorado Springs Commandery, Knights Templars; interment was in Evergreen Cemetery, Colorado Springs, Colo.

He was married in Poplar Bluff, on February 14, 1905, to Bessie Flanigan, who survives him.

Mr. Nickey was elected an Associate Member of the American Society of Civil Engineers on November 25, 1919.

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* Memoir prepared by E. C. Thomes, Esq., Poplar Bluff, Mo.

ALBERT EDWARD PADDOCK, Assoc. M. Am. Soc. C. E.*

Died April 20, 1929.

Albert Edward Paddock was born at Malone, N. Y., on April 15, 1884. After completing a public school education he entered Syracuse University, at Syracuse, N. Y. He was registered in the Engineering Department, where he completed the first two years of a four years' course. Owing to the death of his father before the completion of his second year at college, he left school, with characteristic unselfishness, to assume the responsibilities of an oldest son of a large family. While at Syracuse he had taken an active part in athletics as Captain of the Freshman Crew and as a member of the University football team. Actuated by a determination to be self-supporting, he accepted in 1906 an assignment as Foreman of Construction and Operation of Water-Works in Dawson, Yukon Territory, where he remained for two years. On returning to the United States, he was employed as a Mechanical Operator for the Northwestern Improvement Company at Cle Elum, Wash.

In November, 1911, Mr. Paddock joined the U. S. Reclamation Service and was assigned as Surveyman and as a Foreman of construction work on the Tieton Project in the State of Washington. He was soon advanced to Chief of Party and then to Engineer in Charge of Engineering and Construction on a Division of this Project.

His next assignment in the Reclamation Service was as Engineer in Charge of Construction on a Division of the Sunnyside Project in Washington, on the completion of which he was assigned to the position of Locating Engineer on the Okanogan Project. Subsequently, he became Assistant Tunnel Superintendent on one of the features of the Yakima Storage Project.

When the Yakima work was completed in August, 1913, Mr. Paddock was employed by the Southern Alberta Land Company as Superintendent of Construction on a large concrete diversion dam and canal head-works on the Bow River, in Alberta, Canada. He transferred from this work in September, 1914, to the U. S. Reclamation Service, North Division, and was assigned as Foreman in charge of concrete conduit, outlet tower, and spillway excavation and construction on the Sherburne Lakes Reservoir Dam of the Milk River Project, in Montana.

Mr. Paddock then was assigned to the Shoshone Project, Wyoming, as Assistant Tunnel Superintendent. In this capacity he was engaged on the enlargement of the outlet tunnel and installation of balanced gates outletting from the Shoshone Reservoir. On the completion of this tunnel he was again assigned to the work on the Sherburne Lakes Reservoir Dam, from which he resigned in 1916 to accept a position as Superintendent of Construction for H. G. Fenton, Contractor, on the repair and extensions of the Sweetwater Dam, near San Diego, Calif.

On July 3, 1917, Mr. Paddock entered the employ of the City of San Diego and was assigned as Field Engineer on the construction of the Lower Otay Dam, which work was being carried on by contract. When the project was taken over by the City of San Diego in November, 1918, for construction by municipal forces, Mr. Paddock was advanced to the position of Superintendent of Construction. On the completion of the Lower Otay Dam in 1919, he was transferred in the same capacity to the Barrett Dam, which was also being constructed with municipal forces by the City of San Diego.

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* Memoir prepared by H. N. Savage, M. Am. Soc. C. E.

Mr. Paddock's outstanding success as a superintendent on heavy construction brought him to the attention of the Utah Construction Company to the service of which he transferred in August, 1920, as Resident Construction Engineer and Superintendent of the Hetch Hetchy Dam which the Company had under contract.

He continued in the employ of the Utah Construction Company continuously from that time until his fatal accident, with the following principal assignments of contract work: Construction Engineer on the Freeman Masonry Weir across the Sacramento River, about 25 miles above Woodland, Calif.; from December, 1923, to February, 1924, Superintendent of Construction on excavation and embankment for roadbed, incident to double-tracking the main line of the Southern Pacific System in the vicinity of Truckee, Nev.; from February to December 1924, Superintendent of roadbed construction on the Natrone Cut-Off in Oregon-California, for the Southern Pacific System; from February, 1925, to October, 1926, Superintendent of Construction of the American Falls Dam across the Snake River in Idaho, an important feature of the Minnedoka Project of the U. S. Reclamation Service.

Before the completion of this work Mr. Paddock was also assigned, in October, 1926, as Superintendent of Construction for the Contractor on the Gibson Masonry Dam of the U. S. Reclamation Service, about 70 miles west of Great Falls, Mont. He also continued to direct the building of the American Falls Dam until its completion early in 1927. At the same time he carried on the construction of the Gibson Dam and continued in charge to the time of his death which occurred on April 20, 1929, while in the performance of his duties as he, with customary aggressive responsibility, considered them to be.

The construction riggers at the Gibson Dam, in moving the concrete-carrying chutes, fouled them against the high concrete hoist towers. Mr. Paddock, arriving on the scene and sensing the conditions and consequent delay, characteristically jumped on top of the concrete bucket then at the base of the tower, and signaled the engine-man to hoist him to where the chute was caught, about 100 ft. above the base of the tower. One of the riggers who had been fixing lines higher up in the tower also had noticed that the chute was caught, and without signaling for the bucket, and unknown to the hoisting engineer on the ground, came down the hoist cable hand over hand to where the chute was caught. Thus, he was inside the tower with his hand presumably on the hoist cable when the concrete bucket, with Mr. Paddock standing on top of it, was being hoisted. The rigger lost his hold on the hoist rope and inside tower frame and fell down inside the tower, a distance of about 60 ft., landing on Mr. Paddock. Mr. Paddock was fatally injured and died within a few hours; the rigger, however, escaped with only a few broken bones.

Throughout Mr. Paddock's professional work he had exemplified executive qualifications of a very prominent type. He was outstanding among the multitude of construction foremen in the loyalty he displayed toward, and received from, his associates and subordinates. This was primarily in evidence in his aggressiveness in automatically responding personally to every apparent requirement to do whatever appeared to him necessary without waiting for others, thereby materially advancing the progress of all types of work under his responsible supervision, although he never discounted or interfered with the duty or work of his subordinates who were effectively carrying on.

Mr. W. H. Mattis, President and General Manager of the Utah Construction Company, writes of the loss sustained by his Company as:

> "* * * immeasurable, as he was a vital part of the Company, and an exponent of its policies. * * * his unusually high place in our organization was due to his outstanding personal ability and character. He seemed to exemplify all the ideals of high-minded men; he had the confidence of his superiors, the affection of his men, and the respect of all with whom he dealt."

On September 14, 1920, Mr. Paddock was married to Mary G. Riveroll, in Los Angeles, Calif., who survives him. He is also survived by his mother, two brothers, and a sister. He was a member of Delta Upsilon Fraternity, a Scottish Rite Mason, and a member of Al Bahr Temple, A. A. O. N. M. S. He was buried in San Diego, Calif.

Mr. Paddock was elected an Associate Member of the American Society of Civil Engineers on November 26, 1918.

CHARLES HENRY REYNOLDS, Assoc. M. Am. Soc. C. E.*

Died November 1, 1929.

Charles Henry Reynolds, the son of Mr. and Mrs. Aldrich J. Reynolds, was born in Forestville, near Manchester, Iowa, on December 3, 1874. At the age of two he moved to Fort Dodge, Iowa, where he resided until his death.

Mr. Reynolds received his education in the public schools of Fort Dodge and was graduated from the High School. After his graduation, he was Assistant to the City Engineer at Fort Dodge for five years, during which time the first pavement was laid in that city. As to his additional education, he studied under Mr. Easley when he was his Assistant City Engineer, and, later, took a correspondence course in Engineering, which he completed with a very high standing.

His entire professional ambitions were wrapped up in the growth of his city. During his long career of thirty-one years as City Engineer, Mr. Reynolds saw the city emerge from its first permanent sidewalk, which he constructed of brick, to the many miles of paving it now possesses. In addition, many beautiful bridges in Fort Dodge are also monuments to his activities. Another improvement which took place under his direction and of which he was exceptionally proud, was the selection of numbers for city streets in place of the names which they formerly had. This change took place about 1900.

Mr. Reynolds was probably more closely identified with the growth of Fort Dodge than any other one person. He took much pride in his native city and was always ambitious for its improvement. The last photograph he had taken was on the site of the new City Water-Works, on North First Street, where he was present to witness the turning of the first shovelful of earth.

In addition to his duties as City Engineer, he was for twelve years County Surveyor of Webster County, and for sixteen years Principal Drainage Engineer in Webster and near-by counties. He also did some general engineering practice for surrounding municipalities for sewers, bridge, and street work.

He was a member and active worker in many lodges and fraternal organizations of the city. Among them were the Elks Lodge, of which he was Past Exalted Ruler and Past District Deputy Grand Exalted Ruler. He was also very active in the Masonic Lodge, having been Past Commander of Calvary Commandery, Knights Templars, Past Patron of the Eastern Star, Past Watchman of Shepherds in the White Shrine, and a member of Za-Ga-Zig Shrine, Des Moines, Iowa.

Mr. Reynolds was a member of the National Arbitration Commission. He was also a member of St. Mark's Protestant Episcopal Church, of Fort Dodge, and, for nine years, was Secretary of the Oakland Cemetery Association, in which office he was responsible for the adoption of the perpetual-care plan, which has been one of the outstanding improvements in the city.

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* Memoir prepared by C. H. Currie, M. Am. Soc. C. E.

A lasting tribute to Mr. Reynolds may well be summed up in the fact that his appointment and re-appointment as City Engineer over a period of thirty-one years were never questioned; and this occurred in a city where politics and officers changed frequently.

Mr. Reynolds was very highly respected among engineers throughout the State and by every one who knew him. He was married in 1898 to Laura Ellen Beresford, of Fort Dodge. Besides his widow, he is survived by his son, Jack, and two daughters, Myriam and Helen.

Mr. Reynolds was elected an Associate Member of the American Society of Civil Engineers on November 25, 1919.

HARRY CHARLES SANFORD, Assoc. M. Am. Soc. C. E.*

Died April 22, 1928.

Harry Charles Sanford, the son of Sylvan and Julia (Sanford) Sanford, was born at Mantua, Ohio, on December 29, 1869. His father, who was of English descent, was a highly respected doctor of medicine.

Mr. Sanford received his early education in the schools of his native town, later attending the Grand River Institute at Austinburg, Ohio, for two years, and completing his education at Valparaiso University, Valparaiso, Ind., from which he was graduated in 1889 with the degree of Bachelor of Science.

His first work was in Cleveland, Ohio, where for several months, beginning in August, 1889, he was engaged as Draftsman on decennial maps of that city. From May, 1890, for a year, he was Rodman, Transitman, and Assistant to the Division Engineer, on the Akron and Chicago Junction Railroad (now a part of the Baltimore and Ohio System). From 1891 to 1895, Mr. Sanford was Assistant Engineer to Ryan and McDonald, Contractors for the construction of the Baltimore Belt Line Tunnel, which is used by all passenger trains on the Baltimore and Ohio Railroad passing through the City of Baltimore, Md. On this, his first experience with contractors, he found the work so congenial that it determined his life career.

During 1895, Mr. John B. McDonald was awarded the contract for the construction of the Jerome Park Reservoir, the large storage reservoir for the Croton Water System located in the Borough of The Bronx, New York, N. Y.; it was to occupy the site of the Jerome Park Race Track. Mr. Sanford transferred to this work and remained on it for approximately two years, in charge of plant layout and construction plans. In 1897, although the contract was far from completion, Mr. Sanford resigned his position and took another with Rodgers, Farrell and Hegeman, Contractors for the construction of twelve miles of the Pittsburgh, Bessemer and Lake Erie Railroad, a subsidiary of the United States Steel Corporation. In less than a year he was made Resident Engineer on the same railroad, in charge of the construction of a steel viaduct, 4 000 ft. long, and of 3 miles of heavy grade work. For about one year he was Division Engineer with the Buffalo, Rochester, and Pittsburgh Railroad Company during which time 10 miles of construction were completed between Craigsville and Butler, Pa.

Early in 1899 Mr. Sanford joined the forces of the Degnon Contracting Company. In December of that year the contract for the construction of the first Rapid Transit Railroad (underground) in New York City was awarded to Mr. John B. McDonald. This work, involving twenty-one miles of structure, was divided into "sections" which were carried on by sub-contractors, of which three sections in Manhattan were awarded to the Degnon Contracting Company. Two of these were contiguous from City Hall to Bond Street, and the third was along 42d Street and Broadway to 50th Street. Mr. Sanford was placed in charge of the latter section, which was without doubt the most difficult piece of city construction ever undertaken. On both 42d Street and Broadway heavy street cars were in operation, supplied by underground trolley current; large water and gas mains had to be maintained in service; many high-tension and telephone wires and cables were in conduit which had to be broken away; the wires and cables were carried on the street timbering and the sewers in flumes.

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* Memoir prepared by William J. Boucher, M. Am. Soc. C. E. Mr. Boucher died on September 14, 1929.

The underlying material everywhere was rock, which had been excavated only sufficiently to permit the placing of these utilities. Therefore, all the space required for the four-track railroad and stations had to be excavated in rock below all these services. Practically all the buildings alongside the railroad had vaults under the sidewalks, in which were heating or electric plants or storage space; taking over the space occupied by these vaults involved bulkheading, re-arranging, etc., all a very tedious operation. At Park (Fourth) Avenue and 42d Street, and at 42d Street and Broadway, the railroad curved under private property (obtained under easement). At the former of these locations the construction of the Belmont Hotel was carried on simultaneously with the construction of the railroad; similarly, at the latter location, the construction of the New York Times Building was proceeding; in both buildings the columns were ingeniously placed between the subway tracks.

When it is realized that there were no precedents to guide the engineers or contractors in this undertaking, and that the work was done in rock and under city streets with heavy continuous traffic, and with only negligible delays or incidents, its magnitude grows more and more and reflects great credit upon all concerned and particularly upon Mr. Sanford. On the completion of this work, in 1904, the Degnon Company was re-organized and Mr. Sanford was made Chief Engineer; he retained this position as long as the Company remained active (until 1921). He also filled the office of Treasurer of the Company from 1904 to 1917.

During 1904, 1905, and 1906, the construction of the subway was carried on under Broadway (Manhattan), from St. Paul's Church to Bowling Green. This work, the first under a wood deck street carrying traffic (a system originated, and here first applied, by him) was Mr. Sanford's responsibility.

From 1906 to 1909 the Degnon Company was engaged, under the direction of Mr. Sanford, in constructing a section of the Center Street Subway, which is now operated by the Brooklyn-Manhattan Transit Company, and the Sixth Avenue Subway from 12th Street to 33d Street, which is operated by the Hudson-Manhattan Railroad Company. This latter was excavated entirely in rock, the elevated railroad columns being supported along the entire distance and, later, their load being transferred to the roof of the reinforced concrete subway structure.

From 1909 to 1912, Mr. Sanford was in charge of the construction of that section of the Catskill Aqueduct known as "Contract No. 47", in Ulster County, New York. This covered the south half of the Bonticou Grade Tunnel, 0.65 mile; 3.8 miles of cut-and-cover aqueduct; the short Mohonk Grade Tunnel; and the Wallkill Pressure Tunnel, 4.5 miles long—a total of 9.25 miles. The work, as the names indicate, was varied in character, with heavy earth and rock cuts and much deep tunneling. Six working shafts were required varying in depth from 348 to 482 ft. The equipment was composed of two compressed air plants, one electric power plant, and a rock-crushing plant. A 36-in. gauge railroad, crossing a special steel truss bridge, served the various sections of the contract. The shafts were electrically operated. The work was carried on expeditiously and efficiently and at what is believed to be the record speed in America to that time for hard rock tunnels of this size; 523 ft. were excavated in one month, the dimensions of the rock cut being 17 ft. high and 13 ft. 4 in. wide.

In the summer of 1912, the Degnon Company began its work on the enlarged subway system of New York City by being awarded a contract for building a section on Broadway, from City Hall to White Street. This was rapidly followed by two sections under Fourth Avenue, Brooklyn; and, somewhat later, two sections under West Broadway and Varick Street, of the "West Side Line"; then a combined subway and elevated link (Route 50) in Long Island City; next, a section along 59th Street and 60th Street, crossing under Fifth Avenue; then a mile in Brooklyn known as the Flatbush Avenue Tunnel, a two-track line built through sandy ground entirely by tunneling methods for which a specially designed roof shield was used, concrete being placed by pneumatic pressure. Another difficult section was under St. Felix Street, and the Long Island Railroad Station, in Brooklyn; and, finally, a section was constructed under 14th Street, extending east from Union Square, Manhattan, which made a total of ten sections in a period of somewhat more than five years—a record equalled by no other single subway contractor. All this work, aggregating in cost more than $20 000 000, came under the direct supervision of Mr. Sanford, who had the responsibility for making the estimates and bids in obtaining the work, and, later, in carrying on the construction.

During 1914, the Public Service Commission announced its plans of extending the 59th Street line to join the Boroughs of Manhattan and Queens by laying two additional tracks on the lower or roadway level of Queensborough Bridge. The Degnon Company had large real estate interests in Queens and one of the attractive features of the property was its close proximity to Manhattan markets, which could easily be reached by means of motor-truck distribution—then rapidly becoming popular. The plan to place two additional tracks on the bridge would, of course, necessitate taking about 30 ft. from the roadway, thus seriously reducing the capacity, which, even then, was very undesirable. The Degnon Company, on Mr. Sanford's initiative, made protest against this taking of part of the roadway, and Mr. Sanford, assisted by the late William J. Boucher, M. Am. Soc. C. E., made a complete investigation of the possibility of a tunnel under the East River at the extension of 60th Street.

As a result, the Degnon Company made a proposition to the Public Service Commission offering to build the tunnel for a definite fixed lump sum of money and to accept a substantial reduction on its 59th Street Subway section. The plans were changed accordingly and bids were asked for the construction of two single-track tunnels. The Degnon Company's figures were made public and were recognized as the maximum that could be asked and when the bids were opened it was found that another contractor had made an offer about $26 000 lower. As is known, the tunnels were built, thus saving the full roadway of the bridge for vehicular traffic, which now has reached such tremendous volume that plans are under way (1929) to increase its capacity by re-locating the elevated tracks and trains on the upper level.

During 1914 and 1915, Mr. Sanford, as Vice-President of the New York and New Jersey Construction Company, a subsidiary of the Degnon Company, supervised the construction of the two-mile rock tunnel of the Passaic Valley Sewer, built at a depth of about 250 ft. below the surface of Newark Bay, New Jersey. This work, with Reginald H. Keays, M. Am. Soc. C. E., in immediate charge, was excavated through rock and lined with concrete; the rate of excavation per month set a new record for rock tunnels.

When the United States entered the World War in 1917, the Degnon Company offered the services of its organization to the United States Government for the construction of a portion of the many projects which were undertaken. Contracts for demolition and excavation in the Brooklyn Navy Yard and for the construction of warehouses and other buildings, were finished in 1917. Several months were occupied in constructing the launching ways for fabricated steel vessels at the yards of the Submarine Boat Company on the west shore of Newark Bay. Here, thousands of wooden piles were placed in the extremely cold winter of 1917-18, and a record for pile-driving was established when 82 were driven by one crew in one working day. To all this work Mr. Sanford gave his attention and earnest thought.

In May, 1918, the Degnon Company, as co-contractor with Mason and Hanger, was awarded the dock and inland warehouses of the Charleston, S. C., Port Terminal—a rail and water project for the receipt, storage, and embarkation of supplies of all kinds to the war front in Europe. The two organizations labored side by side from May to December and, after the Armistice had been signed, the small part remaining unfinished was completed by the Mason and Hanger Company. Throughout the seven months of activity, Mr. Sanford, known as General Manager of the enterprise, and his wife, made Charleston their residence. Every day, many times long into the night, Sundays included, he devoted his full time and energies to the work, and by his untiring devotion he inspired all who were with him to give their best efforts to the success of the project.

On returning to New York, in 1919, Mr. Sanford was kept busy straightening out some of the tangles which were inevitable, due to the demands of the war. In June of that year he resumed work on an uncompleted sewage treatment contract in Philadelphia, Pa., on which construction had been started in 1917, but had been discontinued at the request, or demand, of the United States Government. Two highway contracts, also in Pennsylvania, were undertaken and carried out between 1919 and 1921.

On the completion of these latter contracts, Mr. Sanford took a much needed rest. For two years he devoted the greater part of his time to a project very dear to his heart—commercial apple orchard work in Ulster County, New York. There, several years before, he had planted hundreds of choice varieties of fruit, which were just then coming into bearing. He maintained a home in Englewood, N. J., but he and his wife spent a large part of their time at the orchards, although he was also frequently called in consultation of engineering matters.

In November, 1923, Mr. Sanford was engaged by Gibbs and Hill, Consulting Engineers, as Engineer-in-Charge of the construction of the power house at Narrows, Va., and of the pole line for the electrification of 134 miles of the Virginian Railway. This work occupied about one and one-half years and was not an easy task, for the foundation conditions proved very troublesome, requiring considerable ingenuity and ability in its execution to completion.

During March, 1925, Mr. Sanford began the work which was destined to be his last engagement. The Rosoff Subway Construction Company made him its Chief Engineer and Vice-President on the first contract it obtained on the Eighth Avenue Subway in New York; this contract covered the section extending from 122d Street to 132d Street on St. Nicholas Avenue. Somewhat later three other contracts on this same line were awarded that Company, making a total of two miles of subway. In this capacity, Mr. Sanford was again on work with which he was thoroughly familiar and the progress was extremely rapid; as was to be expected, he gave the best that was in him. His duties were many, his responsibilities great, his working hours long, and his recreation periods far apart. The result of this overwork was inevitable—he was stricken with a cold in April, 1928, which quickly developed into pneumonia, from which he failed to rally. The end came on the 22d of that month. He had made many plans intending to retire from active work in a comparatively short time.

Mr. Sanford was a quiet man—he preferred to listen to what others had to say. When he came to a conclusion, it was a sound one. Slow to admit any to his friendship, he was apparently weighing those whom he met; but when he knew his friends and had tried them, he opened wide his heart and his home to them. His acquaintances were legion, those who were privileged to call him their friend were comparatively few. It was indeed a privilege to know him; all feel his loss keenly. He was essentially a "home man", and he and Mrs. Sanford were generally to be found there, where a cheery welcome always awaited the caller.

Mr. Sanford was for many years a member of the Methodist Episcopal Church and for some time had been on the Official Board of St. James Church, New York City. On February 27, 1901, he was married to Helen Kurtz, of Cleveland, Ohio, who survives him. They had no children.

Mr. Sanford was elected an Associate Member of the American Society of Civil Engineers on October 5, 1898.

ERICH GEORGE SCHROEDER, Assoc. M. Am. Soc. C. E.*

Died September 24, 1928.

Erich George Schroeder was born on September 15, 1886, in Templin, near Berlin, Germany. The youngest of ten children he was only four and one-half years of age when his family emigrated to the United States, their destination being Milwaukee, Wis., where two older brothers, William R. and Paul, already had settled.

Mr. Schroeder's early boyhood was spent in Milwaukee, where he went to school and, later, was employed as a steam-fitter and plumber with Schroeder and Suelflow. After a year or more he entered Marquette University and, subsequently, the University of Wisconsin, at Madison, Wis., from which he was graduated with the degree of Bachelor of Science in Civil Engineering in 1914.

Immediately after his graduation, he was appointed Chief Estimator and Designer of Paul Riesen's Sons, Contractors, with which Company he remained until March, 1915, when he resigned to go to Clarksburg, W. Va., in charge of the Design and Construction Department of the Concrete Steel Bridge Company. In October, 1915, he and his friend, Abbey, opened offices, under the name of Schroeder and Abbey, in Philadelphia, Pa., as Contractors, Designers, and Architectural Engineers. This business association, however, did not seem to prosper and, in consequence, they dissolved partnership and Mr. Schroeder returned to the West.

In February, 1916, he again located in Milwaukee and opened offices for the Portland Cement Association, of Chicago, Ill., as Division Engineer for Wisconsin. In this position he traveled over the entire United States engaged in general promotion work for cement, especially its use in road building. He addressed public meetings and acquired by experience an excellent knowledge of salesmanship and the ability to deal with public officials and contractors. In December, 1916, the Paul J. Kalman Company, of St. Paul, Minn., sent Mr. Schroeder to Omaha, Nebr., as Manager of a branch there, in which capacity he designed and estimated reinforced concrete buildings. In August, 1917, he was taken ill in St. Paul and was forced to return to Milwaukee where he remained for a time, gradually recuperating until March, 1918, when he accepted a position with the Truscon Steel Company, of Milwaukee, to design and estimate concrete structures.

In July of the same year, however, Mr. Schroeder was on his way across the "great pond" to serve his country, with the 26th Engineer Regiment, in France, but he did not see much active service, as the Armistice was signed on November 11, 1918. He attended the field university in L'Ans where he took a course in Architecture, and made a few trips to Paris and to Southern France, Nice, and the Riviera.

On his return from overseas in July, 1919, he was appointed by the Concrete Engineering Company, of Omaha, as Manager of its Milwaukee Office.

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* Memoir prepared by William R. Schroeder, Esq., Milwaukee, Wis.

From July, 1920, until March, 1922, he was associated with the Robert Reisinger Company as Estimator, Designer, and Superintendent of Construction and, in 1922, he re-opened his own offices in Milwaukee as President of the Erich G. Schroeder Company. He designed buildings, bridges, etc., doing special work in the erection and design of apartment houses. Relative to this subject one of his friends remarked at his death: "He is no more, but his buildings will be his monuments". Among them are the Ellison Apartment, the Miramar, Seville, Sereno, Wesco, Alberta, Riviera, Oak Crest, Oak Park, Oak Manor, Sheldon, and Miami Apartments, and his own—the last—the Belleview Manor Apartment.

This latter building was just completed when Mr. Schroeder became ill on September 18, 1928. He was taken to the hospital on September 19, where he underwent an unsuccessful operation for appendicitis. He died on September 24, 1928, when only 42 years of age. He is survived by his widow, Marie J. Schroeder, four sisters, Rosalie M., Lydia P., Ella L., and Elizabeth H. Schroeder; one brother, William R. Schroeder, all of Milwaukee, a nephew, William F. Schroeder, of Evansville, Ind., and two children whom he had adopted on Mother's Day, 1928, at the ages of one year, Nancy R. and Erich George Schroeder.

Mr. Schroeder was well liked by his friends, his contractors, his employees, and his old college and university chums. They always will remember him for his happy disposition, his good humor, his jokes at the opportune moment, and last, but not least, his great skill on the football field. As his father died in 1897, and his mother in 1902, his older brothers and sisters were father and mother to him, taking care of his education which involved sacrifices he never forgot nor failed to increasingly appreciate.

He was a member of the Alpha Gamma Phi Fraternity, the Milwaukee Athletic Club, the Elks Club, and the American Legion.

Mr. Schroeder was elected an Associate Member of the American Society of Civil Engineers on August 28, 1922.

THOMAS PATTON STEVENSON, Assoc. M. Am. Soc. C. E.*

Died November 8, 1928.

Thomas Patton Stevenson was born in Philadelphia, Pa., on May 6, 1876. In 1897, he matriculated at the University of Pennsylvania where he studied Civil Engineering for two years.

In May, 1900, Mr. Stevenson went to Cuba as Assistant with the Engineer Corps of the Cuban Steel Ore Company on the construction of ten miles of railroad; from August, 1900, to February, 1902, he was engaged with the Engineering Department of Santiago, Cuba, (under Capt. S. D. Rockenbach, U. S. Corps of Engineers, Engineer Officer, and the late A. S. Hobby, Jr., M. Am. Soc. C. E., Principal Assistant Engineer) in surveying, road and culvert construction, and sewer work.

In 1902, he organized the firm of Stevenson and Headman, Engineers and Contractors, of Philadelphia, Pa., and Rio de Janeiro, Brazil. In March, 1902, the firm laid by contract 4 000 m. of pipe and connection with stand-pipe in the City of Paramaribo, Dutch Guiana; in 1905, it built by contract 6 km. of railroad embankments, 100 km. in the interior of Dutch Guiana on a system known as "Suriname Koloniale Spoorwegen", and also signed a contract with the "Koloniale Spoorwegen" for railroad ties (native wood).

Having liquidated his firm, Mr. Stevenson went to Para, Brazil, in 1906, as the Construction Representative of an American Syndicate holding concessions there. In this same year, he went to Rio de Janeiro where he held a position as Assistant Engineer in the Rio de Janeiro Tramway, Light, and Power Company, which had just been organized with the object of supplying electric tramway power and light service to that city. He remained in this position until 1912 when he founded the Federal Express Company in Rio de Janeiro, of which Company he served as President until his death.

During the last sixteen years of his life, Mr. Stevenson demonstrated an extraordinary capacity for work and organization. He led the Federal Express Company which he formed with American and Brazilian elements and capital to sound prosperity, thus giving an example worth imitating, with respect to efficient co-operation of American interests in the development of Brazil.

The Federal Express Company, which continues its successful operation under the administration of his remaining associates, enjoys a very high standing in business circles and is the Agent in Brazil of the Munson Steamship Line, the American Express Company, the White Company, the Marine Office of America, and many other important American firms. In addition to representing American interests, the Company conducts on its own account a general express, transportation, storage, insurance, tourist, and banking business.

Mr. Stevenson was very active and had an extraordinary amount of executive ability which he utilized in his social as well as in his business relations. He was not only one of the founder members of the American Chamber of Commerce for Brazil, but was also its first Secretary. He served continuously and tirelessly in this capacity until after many years he was induced to accept the Presidency. He was also one of the founders of the Rio de Janeiro Country Club of which he served as Director and he was a member of the Jockey and Golf Clubs. His vision and initiative in establishing the Country Club made evident only one of the many ways in which he used his great ability for the pleasure as well as the profit of the entire community. He was deeply interested in furthering good relations between Brazilians and Americans, and he worked to this end with great success.

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* Memoir prepared by A. Lund, Director, The Federal Express Co., Rio de Janeiro, Brazil.

Mr. Stevenson died of heart disease, from which he had been suffering for a long time, at his home in Rio de Janeiro on November 8, 1928. His burial was attended by practically the entire American colony and by the most prominent members of Brazilian society and of business firms of Rio de Janeiro. He is survived by his widow, Esther S. Stevenson, of Philadelphia, Pa.

His success in Brazil was due, to a great extent, to the fact that he made Rio de Janeiro his permanent home, took pains to study and understand the psychology of Brazilians, and to follow the Republic both in its material and intellectual aspect. He associated freely with Brazilians and, consequently, was able to promote the American business interests with which his Company was connected in a most efficient manner and without the friction which develops when there is lack of mutual understanding.

Mr. Stevenson was elected a Junior of the American Society of Civil Engineers on April 4, 1905, and an Associate Member on November 6, 1907.

HARRISON STIDHAM, Assoc. M. Am. Soc. C. E.*

Died July 9, 1927.

Harrison Stidham, the son of Alfred D. and Martha J. (Gorham) Stidham, was born on April 6, 1869, in Washington, D. C. He attended the public schools of Washington and was graduated from the Central High School in 1887. He entered Cornell University at Ithaca, N. Y., in 1887 and was graduated with the Class of 1891, receiving the degree of Civil Engineer.

Following his graduation from the University, Mr. Stidham was employed from 1891 to 1894, with the United States Coast and Geodetic Survey in connection with the primary triangulation along the 39th Parallel, Kansas and Colorado, and the survey of Pensacola Bay, Florida. In June, 1894, he was employed by the late George E. Waring, Sanitary Engineer, in the construction of sewers and sewage disposal systems. In 1895, he went with Colonel Waring as District Superintendent, in the Department of Street Cleaning, New York City; later, he became Superintendent of Snow Removal and Assistant General Superintendent, and was detailed by Colonel Waring to investigate new appliances and methods.

In April, 1898, Mr. Stidham was appointed General Superintendent of The Tubular Dispatch Company of New York. This Company operated an 8-in. tube system carrying United States mail in New York. During the period of Mr. Stidham's connection with the service the extension of the system was made across Brooklyn Bridge.

In January, 1900, he entered consulting practice as a Sanitary Engineer in New York City in partnership with Mr. George L. Walker. During the period, 1901 to 1905, he was with the American Bridge Company, in the Division Engineer's Office, New York, associated with the Department of Estimates and Costs.

In January, 1905, Mr. Stidham returned to Washington as Superintendent of the Street Cleaning Department. During the following year he re-organized the Department and placed it on a highly efficient basis. At that time the Washington Fertilizer Company held the contract for the disposal of garbage in the District. Its service was unsatisfactory and wholly inadequate for the standards of the Street Cleaning Department, as set by Mr. Stidham, and, during 1905, the Company was heavily penalized for failure to meet requirements. In January, 1906, the Company retained Mr. Stidham as General Manager, and under his efficient direction it was re-organized on a basis that afforded satisfactory service and a profitable return. Mr. Stidham continued as General Manager until January, 1918, when the Company was taken over by the United States Government.

During the World War, Mr. Stidham was a Captain in the Construction Division, stationed in Washington. Following the war he became Secretary-Treasurer of the Wilkins Security Company and thereafter devoted himself to business affairs. His faculty for organization and business management combined with his technical ability, brought marked success to his various activities. He had a lively interest in club work and public affairs and left a host of friends in the numerous organizations with which he was connected.

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* Memoir prepared by James H. Edwards and John C. Hoyt, Members, Am. Soc. C. E. Mr. Edwards died on August 14, 1930.

Mr. Stidham was the author of "Problems of Snow Removal" (1896), and "Pneumatic Tubes" (1900). He was a member of the Delta Upsilon Fraternity; the Cosmos Club of Washington; the Chevy Chase Club of Maryland; the Kirkside Club of Washington; and the Washington Society of Engineers.

He was married to Clara Kerr, of Collins, N. Y., on September 7, 1901. Mrs. Stidham, with two sons, Alfred and Shaler, and one daughter, Sara, survives him.

Mr. Stidham was elected an Associate Member of the American Society of Civil Engineers on April 4, 1900.

DAVID THOMPSON, Assoc. M. Am. Soc. C. E.*

Died April 29, 1929.

David Thompson was born in Philadelphia, Pa., on March 17, 1887. He was the son of David S. Thompson, a member of the Philadelphia Stock Exchange, and Mary (Taylor) Thompson, the daughter of John M. Taylor, a wholesale dry goods merchant of Philadelphia.

Mr. Thompson's early education was acquired in the public schools and High School of his native city, after which at the age of 17, he entered the University of Pennsylvania. He was graduated therefrom in June, 1908, with the degree of Bachelor of Science in Civil Engineering, and the marked distinction of being the youngest member in his class.

His first professional engagement was from 1908 to March, 1910, as Draftsman on the design of track elevation, etc., in the Grade Crossing Division of the Bureau of Surveys of Philadelphia. During March of the same year, he served as Rodman with the New York Division of the Pennsylvania Railroad Company, at Jersey City, N. J.

Attracted by a higher salary, he accepted in April, 1910, a position with the Isthmian Canal Commission at Culebra, Canal Zone, Panama. He was assigned to the Construction Department and served as Inspector of Dredging, with the Division of Lock Design, and Draftsman on machinery equipment, until October, 1912.

Subsequently, Mr. Thompson was engaged in the following positions: From March to September, 1914, on inspection, construction, and installation of ice-making equipment and refrigeration for the Railway and Stationary Refrigerating Company (The Clothel Company), of New York, N. Y.; from April to October, 1915, on steel inspection under Mr. Lyddon, of the Inspection Office of the English Army, at Pittsburgh, Pa.; from January to March, 1916, with the R. H. Beaumont Company, of Philadelphia, as Draftsman on the design of elevating and conveying equipment; from March, 1916, to July, 1917, in the offices of the Chief Engineer and Assistant Engineer of the Pennsylvania Railroad Company, at Philadelphia, as Draftsman on design and track work; from August, 1917, to September, 1918, during the World War, with the Bureau of Yards and Docks of the United States Navy Department, at Washington, D. C., as Inspector on construction of armor plate and ordnance and projectile plants at Charleston, W. Va., and at the League Island Navy Yard in Philadelphia; and from September to December, 1918, with the Emergency Fleet Corporation of the United States Shipping Board, Department of Shipyard Plants, at Philadelphia, as Draftsman on the design of water pipe lines for fire protection. At this time, Mr. Thompson was compelled to give up active work on account of ill health from which he had suffered ever since his residence in the Canal Zone. By January, 1925, he had recovered sufficiently to continue his professional career, and he accordingly accepted a position in the U. S. Engineer Office at Wilmington, Del., as Inspector on dredging work, where he continued until April.

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* Memoir prepared from information on file at the Headquarters of the Society.

The West seemed to attract Mr. Thompson and in May, 1925, he went to Los Angeles, Calif., where he was active in the City Engineering Department, as Junior Civil Engineer, on street improvements, paving, etc. At intervals, dating from 1912, Mr. Thompson was also engaged in private work.

At the time of his death he was again employed in Government work and was located at Hawthorne, Nev., as Associate Engineer with the U. S. Naval Ammunition Depot at that place. His death occurred, after a short illness, from pneumonia. He is survived by his mother, Mrs. Mary Taylor Thompson, and two sisters.

A gentleman of staunch character and high principles which were the outward expression of an intrinsic fineness of nature, and intensely interested in his profession during his entire career, Mr. Thompson was conscientious, capable, and dependable. He was an affectionate and devoted son, and was highly regarded by his teachers and loved by his classmates. The following tells but slightly of an affection for him which lasted through the years:

> "I remember him as if it were yesterday, as we sat together our four years in college. During our association I became very fond of David, as he was a man of upright character and fine bearing and I feel it was a privilege to have been so close to him during that time."

He was a member of the Methodist Episcopal Church. He had joined that church when he was 12 years of age and had remained active in its affairs throughout his life.

Mr. Thompson was elected an Associate Member of the American Society of Civil Engineers on June 6, 1927.

BOYD FREEZE WALKER, Assoc. M. Am. Soc. C. E.*

Died December 16, 1929.

Boyd Freeze Walker was born in Des Moines, Iowa, on February 3, 1888. He attended the public schools and was graduated from the Des Moines High School. He then entered the Civil Engineering Department of Iowa State College, at Ames, from which he was graduated in 1912 with the degree of Bachelor of Science in Civil Engineering. He was an enthusiastic participant in athletics in High School, and remained interested in them throughout his life. He held the championship in hurdling and running at Iowa State College and also was considered one of the best "high jumpers".

Mr. Walker's first engagement was as Road Engineer for the Chicago, Rock Island and Pacific Railway Company, which position he soon resigned to go to Great Falls, Mont., where he was employed by the Ambursen Company as Construction Engineer on the Great Falls Dam. On its completion he was sent to Prince Albert, Ont., Canada, as Assistant Engineer on the construction of a large dam. From May, 1913, to May, 1914, he served as Deputy County Surveyor at Fort Benton, Mont., on office and field work on road construction.

In April, 1915, he went to Council Bluffs, Iowa, where he secured a position as Assistant to the County Engineer of Pottawattamie County, on road and bridge work. At this time he also undertook his first drainage project in this County. Mr. Walker served as County Engineer of Pottawattamie County for three years, at the expiration of which time he entered private practice, specializing in county drainage work, special investigations, and reports for corporations and individuals, etc., in his own and the surrounding counties, and establishing himself as one of the most capable engineers in that part of Iowa.

In the fall of 1925, a delegation of business men of Council Bluffs called on Mr. Walker asking him to consider the nomination for City Engineer. After debating the question for some time, he accepted the proposal and in March, 1926, he was elected to that office by a large majority. This position he held until his death from anemia after an illness of two months.

Mr. Walker's memory is perpetuated in Council Bluffs by many city improvements which he forwarded. He kept himself well informed on all technical and other subjects, and having been called on repeatedly to serve as Consulting Engineer by various railroad companies in their respective lawsuits, he was studying law at the time of his death.

He was a man of few words, but when he chose to speak his word was one on which to depend. He was kind to and considerate of his fellow workers and there was no undertaking too insignificant for his personal attention. He was an active worker in the various civic organizations of Council Bluffs and was always glad to give his time for any plan that would improve or better his city or State. A warm-hearted, law-abiding citizen, he occupied a most difficult position, but never at any time did he usurp his power or do that which could cause him to be criticized as unfair or unjust.

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* Memoir prepared from information on file at the Headquarters of the Society.

Mr. Walker was a Thirty-second Degree Mason, member of the Shrine and of the Eastern Star, as well as of the Benevolent and Protective Order of Elks and the Beta Theta Pi Fraternity. He was also affiliated with the Iowa Engineering Society, of which he was a Director at the time of his death, and the Engineers' Club of Omaha, Nebr. He was a member of the First Presbyterian Church of Council Bluffs.

On December 20, 1916, he was married to Marigold C. Robey, of Omaha, Nebr. He is survived by his widow, three children, George, Boyd, Jr., and Marigold B.; his mother, Mrs. A. H. Walker, and a brother, George L. Walker, of Des Moines; and three sisters, Mrs. J. S. Brisco, of Arroyo Grande, Calif., Mrs. D. E. Nichols, of Livingston, Mont., and Mrs. A. H. Hinkley, of Puunene, Maui, Hawaii.

As a sincere appreciation of Mr. Walker, the following may be quoted:

> "A man who is faithful to family, country, God, and fellow men and efficient in the discharge of duties and obligations, is a loss to the country when he departs. The loss is greater when death comes in the prime of life as it did in the case of Mr. Walker. Not half of his normal expectancy in years had been lived, and had he been permitted he would have given good service to his country and his family and community for many years. * * * As the record stands, Mr. Walker's family and friends * * * may well feel pride in his accomplishments for they, like his character, were clean and sound and good."

Mr. Walker was elected an Associate Member of the American Society of Civil Engineers on June 10, 1929.

JAMES JOSEPH WALL, JR., Assoc. M. Am. Soc. C. E.*

Died August 16, 1929.

James Joseph Wall, Jr., the son of James Joseph Wall and Amelia Mary (Jones) Wall, was born on February 24, 1892, at Saginaw, Mich.

After graduating from the Central High School at Duluth, Minn., Mr. Wall entered Cornell University and was graduated from the College of Civil Engineering in 1916. During vacations previous to his graduation, he served as Chainman and Rodman for the Canadian Northern Railways and as Concrete Road Inspector at Superior, Wis., for the Portland Cement Association of Philadelphia, Pa. While at Cornell, Mr. Wall was active as a member of the Sigma Nu Fraternity, Manager of Fencing, and Advertising Manager of the "Cornell Civil Engineer".

In April, 1917, Mr. Wall entered the First Officers' Training Camp, at Plattsburg, N. Y., and served during the World War as First Lieutenant, Officers' Reserve Corps, 6th U. S. Engineers, and as a Captain of Engineers, U. S. Army, in command of A Company, 2d Engineers, Second Division, American Expeditionary Forces. He was cited by General Pershing and Major General Lejeune, of the U. S. Marine Corps, and also by General Buat, of the French Army, for bravery and gallantry, particularly in the rapid launching (in 7 min.) of bridges, under heavy fire, by his Company over the Meuse River during the night of November 10, 1918. These footbridges were completed ahead of schedule and resulted in the elimination of enemy machine gun nests with small losses for the Allies.

After his discharge from the Army, Mr. Wall was employed, successively, in the Sales Department of the Lakewood Engineering Company, Cleveland, Ohio, and as Engineer in charge of road grading on Route No. 3, Whitehorse Pike, New Jersey. From February to June, 1920, he was Assistant Field Engineer on Irrigation Construction for the Barahona Sugar Company, Santo Domingo, and on highway location and construction for the Dominican Republic. Following employment on valuation work by the Savage Arms Plant, Utica, N. Y., and as Engineer on concrete road construction at Freehold and Spring Lake, N. J., in 1921, Mr. Wall became Sales Representative for the Holt Manufacturing Company, which position he held until September, 1923. After being connected with the Portland Cement Association, Pittsburgh, Pa., in concrete products promotion in 1924, he became Sales Representative for the Crescent Portland Cement Company, with headquarters at Dayton, Ohio, which position he held until shortly before his death.

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* Memoir prepared by William L. Havens, M. Am. Soc. C. E.

His health failing, Mr. Wall went to Texas, where he was engaged for a short time on work for the Atchison, Topeka and Santa Fé Railway Company at San Angelo, Tex. He was obliged, however, to give up his work and enter the United States Veterans Hospital at Legion, Tex., where he died of tuberculosis on August 16, 1929.

He had an exceptionally pleasing personality which not only made him popular among his fellow students during his college days, but also served him well in his later employment in sales work. His success as an engineer salesman was no less marked than his bravery as a soldier.

Mr. Wall was elected an Associate Member of the American Society of Civil Engineers on December 14, 1925.

ROBERT BROWN MURPHY WILSON, Assoc. M. Am. Soc. C. E.*

Died December 15, 1928.

Robert Brown Murphy Wilson was born at Chatsworth, Ill., on January 2, 1883, the son of Charles A. and Amelia D. (Levering) Wilson. His father was, for many years, a banker in Chatsworth and was largely interested in real estate in that section of Illinois. He subsequently removed to Chicago, Ill., where he engaged in the stock brokerage business.

His grandfather, Dr. R. B. M. Wilson, was a native of Island Magee, near Belfast, Ireland. He was graduated in medicine from the University of Edinburgh, and directly thereafter came to America. Dr. Wilson settled in Washington, Ill., and was prominent in the practice of his profession throughout that section of the State until his death in 1879. Dr. Wilson's wife was Jane Anthony, of Sherburne, Vt., who when a child moved with her father to Illinois and through her, Robert Brown Murphy Wilson was descended from Josiah Wood, of Middleboro, Mass., who was a soldier in the Revolutionary Army.

On his mother's side, he traced his descent from Wiggard Levering who came to America from Leyden, Holland, in 1685, settling near Philadelphia, Pa., where he purchased land between the Schuylkill River and Wissahickon Creek. A portion of this land is now included in Fairmount Park, Philadelphia, and the Levering family has been continuously represented in this section of the latter city for nearly 250 years.

Mr. Wilson was graduated from the Chicago Manual Training School in 1900, and immediately entered the Engineering School of the University of Chicago with advanced standing; but, at the close of his Junior year, owing to various circumstances, he decided to enter the practical work of his profession and to qualify for his degree by outside study. He subsequently completed all his college requirements, except one subject, and was much commended for his proficient work in Spanish.

Mr. Wilson's first engagement was as Rodman on the construction of the Mobile, Jackson, and Kansas City Railroad in Mississippi, during 1902 and a part of 1903. During the latter year he secured an appointment as Surveyman in the United States Engineer District at Mobile, Ala., and was engaged on hydrographic work in that vicinity for two years or more, except for a period of several months, when owing to the failure of Federal appropriations he was temporarily furloughed and accepted a position as Instrumentman with the Florida East Coast Railway Company. During the latter engagement he was occupied with reconnaissance work through the Everglades and from Key Largo to Key West, Fla.

For the next eight years, until 1913, he followed railroad work in Mexico. His first work there was in connection with the construction of the Colima Extension of the Mexican Central Railway, on which he served as Assistant to the Division Engineer. This Division represented very heavy construction and included both tunnel and bridge work. Subsequently, he was placed in charge of the extension of the Mexican Central Railway from Paredon to Saltillo, in the State of Coahuila, and, still later, of another extension from Marfil to Guanajuato, Mexico.

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* Memoir prepared by David G. Anderson, M. Am. Soc. C. E.

In 1909, Mr. Wilson became connected with the Southern Pacific Railway Company of Mexico, and served as Chief of Party on exploration work on the Yaqui River; Roadmaster on the Sinaloa Division; Assistant Superintendent, Maintenance of Way, on the Sinaloa and Sonora Divisions; and Engineer, Maintenance of Way, for all the Southern Pacific Lines in Mexico. His railroad work in Mexico was of the pioneering type and was attended with many difficulties.

For a time in 1913, he was interested in land development in Mexico, but in the following year he became connected with the Union Carbide Sales Company, as traveling representative, promoting the use of carbide in connection with the oxy-acetylene process.

Shortly after the entrance of the United States into the World War, Mr. Wilson entered the first Officers' Training Camp, at Fort Sheridan, Illinois, on May 13, 1917. He accepted a commission as Captain in the Engineer Officers' Reserve Corps on August 15, 1917, and was assigned to active duty the same day; he sailed for foreign service, September 9, accepted a commission as Major of Engineers on November 12, 1918, returned to the United States, June 27, 1919, and was honorably discharged on July 24, 1919, at Camp Grant, Illinois, while serving with the 311th Engineers.

On the completion of his war service Major Wilson resumed his connection with the Union Carbide Sales Company until 1921, when he became associated with the American Steam Conveyor Corporation, in Chicago. In 1923, he returned to Mexico and to the time of his death was General Manager of the Mina La Prinesa, a silver mine in Chilpancingo, Guerro, Mexico. His death, which was the result of several severe attacks of inflammatory rheumatism affecting the heart, occurred while he was a patient at the Evanston, Ill., Hospital, on December 15, 1928.

Major Wilson was a member of the Wilmette (Ill.), Presbyterian Church and took a deep and sincere interest in its work. During the periods when his professional duties permitted him to reside at home, he devoted himself to a boys' Sunday School class. He was conscientious in all his work, considerate and thoughtful toward others, and always to be remembered for his cheery greetings and for the smiles that brought happiness to those with whom he came in contact. He never married, but was always a devoted son. He represented the finest type of a Christian gentleman, and was a worthy member of his chosen profession.

Major Wilson was elected an Associate Member of the American Society of Civil Engineers on June 30, 1911.

FRANCIS HERBERT WRIGHT, Assoc. M. Am. Soc. C. E.*

Died May 29, 1928.

Francis Herbert Wright was born in Memphis, Tenn., on December 8, 1874, the second son of Col. A. D. Wright, formerly a Confederate Officer, and Mary Ellen (Farrell) Wright. Colonel Wright was a lineal descendant of a titled English officer of high rank, who married a daughter of a Spanish Grandee, Gen. Manuel Cardenas, and established his residence on a Spanish grant owned by his distinguished father-in-law—the present site of San Antonio, Tex. Mary Ellen Farrell was the grand-daughter of Capt. John Sappington, an early Mayor of St. Louis, Mo. John Sappington, the third of his line and name in America, served seven years in the American Army during the Revolutionary War. He was a member of the personal bodyguard of General Washington, at Valley Forge, and when the war ended was a Captain on the Staff of General Lafayette. With Daniel Boone and a company of pioneers he was one of the founders of Boonesboro, Ky., and served a term in the Kentucky Legislature before going to St. Louis where he died in 1807, a man of property.

Mr. Wright was educated in private schools, where he was distinguished because of his proficiency in, and liking for, mathematics. He began his engineering career at a very early age on levee surveys and construction along the Mississippi River. In 1891, he was a Rodman and Inspector for the Board of State Engineers, at New Orleans, La., and also for the United States Engineer Office in that city. Then followed work for various railway companies and two years in Oregon, Washington, and Montana, on railway location and construction.

Mr. Wright was employed at the World's Fair, Chicago, Ill., in 1893, and from 1896 to 1899 was an Inspector, Corps of Engineers, U. S. Army, under Capt. (now Maj.-Gen., *Retired*), Mason M. Patrick. While on furlough, in 1898, he was employed for a time as an Assistant Engineer in the office of the City Engineer of Memphis, Tenn. In 1899, he became an Inspector of dredging at Port Royal, S. C., for the Bureau of Yards and Docks, United States Navy, and also engaged on levee construction at Bird Point, Mo., for the Corps of Engineers, U. S. Army. This work ended in 1900. During 1901, he was employed as a Concrete and Masonry Inspector for the Illinois Central Railway Company, entering private practice when that work was completed. In 1902-1903, he served as Assistant City Engineer of Little Rock, Ark., and Assistant Engineer in charge of sewer construction in Helena, Ark. After a short period as Draftsman on location for the St. Louis-San Francisco Railroad Company, he became, in 1903, City Engineer of Helena, Ark., where he remained until late in 1907.

In 1908, Mr. Wright went to Chicago, Ill., where he served as Sales Engineer for about a year for the Northwestern Expanded Metal Company. In 1909 he was employed as Chief Designing Engineer for Mr. Carl Weber, who was engaged in the construction of reinforced concrete chimneys. He left

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* Memoir prepared by Ernest McCullough, M. Am. Soc. C. E.

Mr. Weber in 1910 to become Chief Engineer for and, later, a partner in, the Julian S. Nolan Company, designers and builders of two-way reinforced concrete and tile slabs. Mr. Wright patented an L-shaped closer for the open ends of tiles used in the Nolan-Wright System and the Company was very successful until the beginning of the World War in 1914, which caused a slowing down of the building industry. With the entry of the United States into the war, in 1917, the Company suspended operations entirely.

Mr. Wright volunteered immediately for service and was recommended for the rank of Captain in the Engineer Officers Reserve Corps, by the Army and Navy officers under whom he had served as a young man. The physical examination revealed a faulty heart, hitherto unsuspected, and he was not accepted. During the war he worked, in the vicinity of New Orleans, in charge of the construction of wharves and warehouses, as an Assistant Engineer for Ford, Bacon and Davis, later doing considerable appraisal work for the same Company, principally for the valuation of alien property. After the Armistice was signed, he served as a Construction Engineer for Dwight P. Robinson and Company, Incorporated, later leaving that Company to become Engineer and Eastern Representative in the New York District for the Hay-Walker Brick Company. He remained in this position for several years until the firm went into the hands of a receiver. During the period he was associated with the Hay-Walker Brick Company, he invented several types of wall tile, the patents being taken out in the names of other men. He was an Estimator for the Longacre Engineering and Construction Company, Incorporated, for a time, and then became New York Representative for a patented door, on which work he was engaged at the time of his death.

Mr. Wright was a man of very friendly disposition, one intensely interested in everything that tended toward improvement in the working conditions, remuneration, and treatment of engineers. He had a warm heart and made friends easily. In 1924, he purchased an apartment in a co-operative apartment house in Sunnyside Gardens, Long Island City, N. Y., where he served for a time as an officer of the Co-operative Corporation as well as being active for several terms as an officer of local property owners associations. Nothing was too difficult or disagreeable for him to undertake for the general good of the community, and his death was sincerely regretted by a large circle of friends.

In politics, he was a Democrat and a member of the Anaroc Democratic Club. He had a deeply religious nature and was a faithful and devout member of the Roman Catholic Church.

In 1900, he was married to Mary Alice Smith, in Arkansas. They had two children, Mary Frances, one of the prize winners in the International Turnverein Convention swimming contests in Berlin, Germany, in the summer of 1928, and James Mayo, who is in business in New York, N. Y. Mrs. Wright died shortly after the war and, in 1923, Mr. Wright was married to Sarah Leion, of Norwich, Conn., who survives him.

Mr. Wright was elected an Associate Member of the American Society of Civil Engineers on November 4, 1914.

JOSEPH LLOYD CAGNANI, Jun. Am. Soc. C. E.*

Died August 2, 1929.

Joseph Lloyd Cagnani, the second son of Enrico and Palmira Zavatoni Cagnani, was born in New York, N. Y., on July 15, 1904. He was educated in that city and was graduated in 1928 from Cooper Union, with the degree of Bachelor of Science in Civil Engineering.

It was during this schooling period in 1922, which consisted mainly of evening-session study, that Mr. Cagnani entered the service of the Cutler-Hammer Manufacturing Company of New York, in the capacity of Draftsman on conveyor layouts. Later, in 1923, he became Estimator, Draftsman, and Designer for the Harris Structural Steel Company. His unusual ability and persevering efforts were quickly recognized and won for him the appointment of Assistant to the Chief Engineer.

Mr. Cagnani was intensely interested in mechanics, and, with the development of the airplane and aeronautics, he was eager to enter that field. Imbued with this one idea, he matriculated for post-graduate work in aeronautics at the Guggenheim School at New York University. His inclination and his conscientious application to this branch made him an excellent student, and, convinced of his physical and mental eligibility, he decided to enter the field.

In October, 1928, he became a commercial pilot for the Barrett Airways, Incorporated, at Armonk, N. Y. As he possessed a pedagogical turn of mind and an entertaining personality, and was also equipped with concrete facts and principles, he succeeded, in the spring of 1929, to the position of Director of the Ground School—a position that he filled with remarkable aptitude and understanding.

Subsequently, on August 2, 1929, Mr. Cagnani passed the examination for Transport Flier, attaining the highest average ever given for that test. Rejoicing in his new achievement, later on the same day he was flying a "joy plane" with two young friends, when motor and wing defects and disabilities caused him to fall 2 500 ft. It later appeared that he had fought death calmly and coolly every foot of the way down, as the inspectors found him with one hand on the ignition switch, to prevent fire, and the other on the controls.

He died as he had lived, a gentleman unafraid. "Big Joe", they called him, and he was big in heart and soul as well as in body. Four qualities go to make a man—courage, strength, loyalty, and intelligence—and all four Mr. Cagnani had in abundance. His death was a loss not only to his friends, but to all who knew him.

Mr. Cagnani was elected a Junior of the American Society of Civil Engineers on October 1, 1927.

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* Memoir prepared by Mrs. Arthur Levin, Lazar Levin, and Charles W. E. Schroeder, Jun. Am. Soc. C. E.

FREDERICK OLIVER MACY, Jun. Am. Soc. C. E.*

Died December 6, 1928.

Frederick Oliver Macy was born on May 10, 1905, at St. Croix Falls, Wis., the son of Elbert Clyde Macy, M. Am. Soc. C. E., and Minta (Tilden) Macy. Owing to the character of his father's business, that of a Construction Engineer, the boy's grade school education was obtained in the States of Washington, New York, and Iowa. A full four-year High School course was completed, however, in Ames, Iowa, at the home of his grandfather, Mr. Fred C. Tilden.

During this time, Mr. Macy showed the same devotion to study and the disposition to think clearly and accurately that characterized his later work in the university and in business. He took an active part in athletics while in High School and twice won the tennis medal. He played each year in the class basketball teams. He was a member of the track relay team in 1923 that took the National High School Medal. He was Athletic Editor of the High School paper during 1922-23, was Advertising Manager during his Senior year, and Manager for the Class Annual for his Class of 1924.

While attending the Ames schools he worked during the summer vacations of 1918 and 1919 as Transportation and Material Yard Clerk in the Hog Island Ship Building Yards near Philadelphia, Pa., and although young, thus served his country during the World War. In the summer of 1919 he worked as a Machinist Helper for the Great Western Power Company, in connection with the building of a power plant in the mountains of California.

From September, 1921, to July, 1922, Mr. Macy accompanied his family to Japan, China, and the Philippine Islands and worked with his father on proposed power developments. He thus began with practical experience a profession that he had chosen to be his life's work.

After finishing High School Mr. Macy entered Leland Stanford University at Palo Alto, Calif., where he took the civil engineering course and was graduated, with honors, in the Class of 1928. During vacations he was, in 1926, again employed at the scene of his earlier work, the enlargement of the Big Meadows Dam of the Great Western Power Company. In the summer of 1927, he was engaged on building construction in Los Angeles, Calif., and won the high esteem of his employers. His work at the University was of the highest order, which, together with his practical experience, gained in vacation periods both while in high school and in the University, won for him the opportunity to serve as Instructor in Surveying while still finishing his own course. During his Senior Year he was elected President of the Student Chapter of the Society and was chosen by his Class to represent it in presenting the bronze plate for the Class of 1928 to be placed in the Memorial Chapel.

Before his graduation, Mr. Macy was offered a responsible position in connection with making surveys and plans for a large water system to supply a group of cities in the neighborhood of San Francisco, Calif. His previous experience with a number of important engineering projects during his vacations was a guaranty of success here. He continued in this work until late in November, 1928, when he resigned to accept a position in connection with power plant work in Europe. While en route from San Francisco he was stricken with pneumonia and became very ill before reaching his grandfather's home in Ames. He was immediately removed to the Mary Greeley Hospital where every possible effort was made to meet the crisis, but on December 6, with his family about him, he slipped away just before noon.

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* Memoir prepared by Charles B. Wing and F. M. Thebo, Members, Am. Soc. C. E.

Few young men have been privileged to look forward to a more promising place of service in his chosen profession and none enjoyed a more complete respect and love than "Ted" Macy. He had a genius for friendship, born of a native grace of manner which was a fit expression for the kindly consideration that he cherished for all his companions.

To know him was to be charmed and won by his frank friendliness and generous thoughtfulness. To share his life was to be stimulated. Had he lived he would have gone far in his profession. He had already advanced far in the high adventure of unselfish friendship.

Mr. Macy was elected a Junior of the American Society of Civil Engineers on October 1, 1928.

LOUIS ALFONZO PEDERSON, Jun. Am. Soc. C. E.*

DIED APRIL 5, 1929.

Louis Alfonzo Pederson, the only son of Mr. and Mrs. H. A. Pederson of Tucson, Ariz., was born at Saint David, Ariz., on June 6, 1903. He was educated in the public schools of Saint David and Bisbee, Ariz., and was graduated from the Saint David High School in 1922 with the highest honors of his class. Through his efforts in his High School career, he earned a scholarship at the University of Arizona.

Mr. Pederson began his engineering career in September, 1922, working as a Chainman for P. E. Fernald, U. S. Mineral Surveyor at Tucson, in which capacity he was engaged until April, 1923. From May to August, 1923, he was employed as Head Chainman for Mr. Fernald.

Mr. Pederson's parents moved to Tucson in the spring of 1923, and Mr. Pederson then attended the University of Arizona from September, 1923, to May, 1924, as a student in Civil Engineering.

In June, 1924, he accepted a position with W. R. Elliott, Consulting Engineer, of Phœnix, Ariz., and was employed by Mr. Elliott until February, 1925, in the capacities of Head Chainman, Rodman, and Draftsman, successively.

From March to June, 1925, Mr. Pederson was with the Beardsley Land and Investment Company, of Phœnix, as Rodman, Instrumentman, Calculator, and Draftsman on irrigation and power investigations in the vicinity of Phœnix, under the direction of A. F. Harter, Assoc. M. Am. Soc. C. E., being in charge of a party on topographic surveys for storage reservoirs on the Agua Fria River. From July, 1925, to June, 1926, he was in the employ of the City of Tucson, as Head Chainman, Instrumentman, and Sand Inspector. He was also employed on city sub-division work and on paving and sewer construction.

In June, 1926, he accepted a position with the Griggs Engineering Company, of Phœnix, assuming charge of all field work on the San Pedro Irrigation Project on the San Pedro River, at Benson, Ariz., under the direction of Mr. Harter. This consisted of all the preliminary surveys on this project, including re-establishment of section corners, topography over the entire area of about 40 000 acres of land, preliminary canal location, location of diversion dam and storage reservoir, bed-rock investigations at the storage dam and diversion dam, and the running of exclusion areas within the limits of the project. Mr. Pederson showed marked ability and was placed in charge of two parties. The field work was completed in April, 1927, at which time he was transferred to Phœnix, where he was employed by the Maricopa County Municipal Water Conservation District No. 1 on canal location and construction connected with the Lake Pleasant Dam on the Agua Fria River. He remained throughout the entire construction period, at all times in direct charge of important work which was completed in March, 1928.

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* Memoir prepared by A. F. Harter, Assoc. M. Am. Soc. C. E.

Mr. Pederson's achievements during the construction period were appreciated to such an extent that on the completion of the undertaking he was retained in the Operating Department as Water Master in charge of the delivery of water to the irrigated areas and, after the irrigating season, he was transferred to the Phœnix Office as Computer and Draftsman. He was employed in this capacity at the time of his death.

His tasks always were carefully and accurately undertaken. Mr. Pederson was one of a group of young, far-sighted engineers who never slighted the work at hand; he was definitely aware and certain of the nature of each problem he attacked before plunging into it, regardless of its difficulty or intricacy. His field and office work stand as a fine tribute not only to himself but to the organization as well.

He was an ardent student at all times, having been enrolled with the Wilson Engineering Corporation, of Cambridge, Mass., and he was well advanced in this correspondence course at the time of his death.

He was unmarried and was affiliated with the Church of Jesus Christ of Latter Day Saints, of which he was a regular attendant.

Mr. Pederson was elected a Junior of the American Society of Civil Engineers on March 5, 1928.

ROBERT BRUETTE POWELL, Jun. Am. Soc. C. E.*

DIED MARCH 11, 1929.

Robert Bruette Powell, the oldest son of Mr. and Mrs. F. J. Powell, of Wausau, Wis., was born at Fort Atkinson, Wis., on May 23, 1900. He was educated at the University of Wisconsin where he received the degree of Bachelor of Science in Civil Engineering in 1923.

During the summers of Mr. Powell's college work he endeavored to gain as much practical experience in his profession as possible. From June to September, 1920, he was employed as Instrumentman with the Wisconsin Valley Electric Company, and in the summers of 1921 and 1922 he served as Inspector and Surveyor on waterway dredging with the Wisconsin Valley Improvement Company. The following year, from March to May, he was engaged in basin and dam site surveys on the Chippewa River for L. A. DeGuerre, Consulting Engineer, Wisconsin Rapids, Wis. At the termination of this short connection, Mr. Powell accepted a position with the Dixie Construction Company, of the Alabama Power Company, and from May, 1923, to February, 1924, he served as Chief of Party on the location of transmission lines and basin surveys.

In April, 1924, Mr. Powell established a connection with the Electric Bond and Share Company, of New York, N. Y., as Designer in the Hydraulic Division. While in this employ he received excellent training and experience in various kinds of work, such as preliminary studies, license drawings, investigation of old structures, substructures, power house and switchyard, penstocks, spillway gate and bulkhead sections, surge tank foundations, and economic studies.

After three years with the Electric Bond and Share Company he was employed by Murray and Flood, Engineers, New York City, and sent to Columbia, S. C., for assignment on the Saluda River Hydro-Electric Development in making hydraulic investigations and other preliminary engineering studies. When the actual construction work began, Mr. Powell was promoted to the position of Supervising Structural Engineer and his main assignment was the supervision of the erection of the penstocks and the intakes. These structures were practically completed before his sudden death in an automobile accident, and now stand as monuments to him.

After his death, letters were received from men with whom "Bob" Powell had worked, and from men who had worked under him, which told frankly of the high esteem in which he was held by those who knew him. One man wrote, "I never worked with one more sincere, more exacting, and more faithful to any trust"; and to those who knew Mr. Powell best these were surely his outstanding characteristics.

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* Memoir prepared by A. R. Wellwood, Res. Engr., Saluda River Hydro-Elec. Development, Columbia, S. C.

T. C. Williams, Vice-President of the Lexington Water Power Company, the owners of the Saluda River Hydro-Electric Development, stated:

"He had such a pleasing personality and was so well liked by every one. I was especially impressed with his cheerful willingness to work just as many hours as necessary, and when there was a peak load, he was always ready to go the limit".

Arthur R. Wellwood, Resident Engineer on the Saluda Development for Murray and Flood, and Mr. Powell's immediate superior, said:

"I feel that I have lost a friend as well as a valued co-worker. He had a very bright engineering mind and I considered him an A-1 Engineer. He was due to go a long way in his chosen field. He had been entrusted with very important features of the construction of the Saluda Dam and had done efficiently each task assigned".

William S. Murray, of Murray and Flood, Engineers, wrote:

"Powell had a compelling personality and I have a very difficult time reconciling the passing out of a man who seemed to have laid all the foundations for an interesting and constructive career".

Mr. Powell was elected a Junior of the American Society of Civil Engineers on April 12, 1926.

CARL SPEER, Jr., Jun. Am. Soc. C. E.*

Died January 18, 1929.

Carl Speer, Jr., the son of Carl and Anna Speer, was born in Baltimore, Md., on December 21, 1901. His preliminary education was received in the public schools of Baltimore. He studied Civil and Sanitary Engineering at the Johns Hopkins University, obtaining the degrees of Bachelor of Science in 1923 and Master of Science in Sanitary Engineering in 1926.

From 1921 to 1922, Mr. Speer was employed as Inspector on construction with the Maryland State Roads Commission. During 1923 and 1924, he served as Junior Engineer with the New York Transit Commission on traffic engineering of subways and elevated lines. From June, 1925, to February, 1926, he was engaged in shellfish investigations for the Maryland State Department of Health. Beginning in July, 1926, and until his death, he was carrying out, as Junior Sanitary Engineer in the Division of Water Safety Control of the City of Chicago, Ill., experimental studies in the treatment of the public water supply.

The writer, as teacher, employer, and friend of Mr. Speer, realizes too well the inadequacy of the foregoing chronological account as a description of an individual of more than usual charm and intelligence. Scientific curiosity and enthusiasm, those rare characteristics of productive men, were possessed by Mr. Speer in an unusual degree. That they would have led him upward to important engineering contributions, his friends and his associates have little reason to doubt. These attributes, coupled with a keen and delightful personality, make his death at the beginning of his career most tragic. The world has lost a promising aide. In all his work, energy and ability were dominant. Ever on the alert for new concepts, he carried on his work with freshness and pleasure. His ideal was to create a value to his fellow man.

On June 26, 1926, he was married to Charlotte A. Stout, of Baltimore. He is survived by his widow, his father, and one sister.

Mr. Speer was elected a Junior of the American Society of Civil Engineers on March 14, 1927.

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* Memoir prepared by Abél Wolman, M. Am. Soc. C. E.

LEWIS IRVING FLETCHER, Affiliate, Am. Soc. C. E.*

Died August 12, 1929.

Lewis Irving Fletcher, the son of Lewis E. and Lucy Ellen (McCracken) Fletcher, was born in Marlborough, Mass., on December 7, 1865. He was educated in the schools of his native town and, later, attended a business college in Boston, Mass.

At the beginning of his professional career, Mr. Fletcher was engaged in construction for the New England Telephone and Telegraph Company, after which he built and operated the lighting plant for the Nashua, N. H., Electric Light Company. In 1889, he again built and operated an electric light and power plant for the Lowell, Mass., Electric Light Corporation.

For a time thereafter, he was engaged in the machinery business in Boston, subsequent to which he went to Easton, Pa., where as Chief Engineer he constructed a hydro-electric plant on the Delaware River for the Easton and Lehigh Power Companies. In 1902 and 1903 he was located at Bulls Bridge, Conn., in connection with the power development on the Housatonic River at that place, and in 1904 he served as Consulting Engineer for the Town of Easton. From 1905 to 1907 he was engaged as Engineer on Dam Construction for the American Pipe and Construction Company of Philadelphia, Pa. During this period he built twenty-two reservoirs and dams along the main line of the Pennsylvania Railroad, in Pennsylvania.

During 1909 and 1910, Mr. Fletcher was employed with the Ambursen Company at Rapidan, Minn., on the construction of a dam and power plant, and with H. M. Byllesby and Company, of Chicago, Ill. Later, he went to Estacada, Ore., where he finished the construction of a dam and hydro-electric plant on the Clackamas River. In 1912, he completed dams and paper mills for the Cornell Wood Products Company, in Cornell, Wis.

He went to Kent, Ohio, in 1913, where he was engaged in general contracting business, building roads and bridges in Northeastern Ohio. From 1921 to 1924, he served as Hydraulic Engineer and Resident Manager for J. Livingston and Company, Inc., of New York, N. Y., at Judsonia, Ark. In 1926 he was engaged on work for the United Gas Improvement Company, at New Milford, Conn., and at the time of his death, he was with the Davey Compressor Company, of Kent, Ohio.

Mr. Fletcher's sudden death at his home in Kent was a shock to the entire community. He became ill as he was preparing to retire for the night and died in less than an hour from an internal hemorrhage. His was a kind and lovable personality, a man who held the marked esteem of scores of friends and associates.

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* Memoir compiled from information on file at the Headquarters of the Society.

Mr. Fletcher was a member of the Masonic Blue Lodge, at Nashua, N. H.; of the Knights Templars and Consistory, at Lowell, Mass.; of the Shrine, at Boston, and of the Order of the Eastern Star and the Benevolent and Protective Order of Elks, at Kent. He was an attendant at the Christian Science Church.

He was married at Lowell, Mass., on October 14, 1890, to Annie A. Shedd, who, with their daughter, Mona, survives him. He is also survived by four brothers, Howard, of Detroit, Mich., Fred, of Marlborough, Mass., Percy, of Fairhaven, Mass., and George, of Newark, N. J.

Mr. Fletcher was elected an Affiliate of the American Society of Civil Engineers on February 6, 1912.

JOHN ROBERT STANTON, F. Am. Soc. C. E.*

Died April 28, 1927.

John Robert Stanton, the son of John and Elizabeth R. Stanton, was born in New York, N. Y., on September 25, 1857. He was educated in the public schools of New York and was graduated from the School of Mines at Columbia University.

Mr. Stanton's career as a Mining Engineer was begun in 1879, in connection with the Atlantic Mining Company and the Central Mining Company of Michigan. In 1890, he became Secretary-Treasurer and Director of the Wolverine Copper Mining Company, and eight years later he was elected Treasurer of the Mohawk Mining Company, of which Company he afterward became President. He also acted as President and Director of the Fort Mountain Talc Company and the White Pine Extension Copper Company of Michigan. His further associations in the field of copper mining were with the Winona Copper Company and the Michigan Copper Company, during which period he was engaged in the active management of eight Lake Superior mines.

Having come from a family of noted metallurgists, Mr. Stanton was especially well posted on the Lake Superior Copper Region. His father had been rated as one of the best informed men on copper in America, and it was estimated in 1905 that the output of the copper mines in which he was interested reached 90 000 000 lb. per year. This fact may account somewhat for Mr. Stanton's interest and accurate knowledge of the business in which he remained active until 1918 when he retired on account of poor health. He left New York to live at his country home at Galesville, Wis., where he died April 28, 1927.

From 1876 to 1887, he was a member of the old Seventh Regiment, N. G. N. Y., later serving six years as Lieutenant and four years as Captain. He was a life member of Company A, Seventh Regiment Veterans Association.

Mr. Stanton was a member of the following societies: The American Institute of Electrical Engineers; the Lake Superior Mining Institution; the Franklin Institute of Philadelphia; the American Association for the Advancement of Science; the National Geographic Society of America; the American Forestry Association; the New York Botanical Gardens Society; the New York Zoological Society; the Horticultural Society; St. George's, St. Andrew's, and the Robert Burns Societies; the Huguenot Society; Sons of the Revolution; the Municipal Art Society; the Thomas Hunter Association; the Society for the Prevention of Cruelty to Children; and the Society for the Prevention of Cruelty to Animals.

He was also active in the following clubs: The New York Yacht Club; the Columbia Yacht Club; the Onigoaming Yacht Club; the Union League Club; the Lotus Club; the Engineers Club; the Republican Club; the Twilight and Dunwoody Country Club; the Chicago Athletic Club; and the Niscowabic Club.

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* Memoir prepared from information on file at the Headquarters of the Society.

Mr. Stanton was married on September 4, 1899, to Helen Maud Kilmer who, with a sister, Mrs. J. W. Moore, of Haughton, Mich., survives him.

Mr. Stanton was elected a Fellow of the American Society of Civil Engineers on April 4, 1899.

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OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

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